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OLDMAN RIVER DAM

OLDMAN RIVER DAM

DESIGN REPORT

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OLDMAN RIVER DAM

DESIGN REPORT

APPENDIX C

DAM AND DYKE
STABILITY ANALYSES

SEPTEMBER, 1988

APPENDIX C

DAM AND DYKE
STABILITY ANALYSES

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C.1 INTRODUCTION

The stability of the Oldman River dam and dyke has been analyzed for both static and earthquake induced dynamic loading conditions. The section of the dam analysed is shown on Plate C-1. Minor design changes to the section have been made since the analysis and the final design section is shown on Plate 3. The section of the dyke is shown on Plate 8.

The conventional two dimensional static limit equilibrium method of slices was used to calculate the safety factors against shear failure of the dam and dyke during the conditions that will exist in the embankment at the end of dam construction, during steady state seepage, during rapid reservoir drawdown, and following an earthquake. The last condition was analyzed using a pseudo-static seismic simulation of the earthquake loading.

A static non-linear finite element analysis of the stresses that will exist at the end of dam construction was done to examine the stress transfer between the core and the shells of the dam, to determine if hydraulic fracturing of the core might occur, to check that no continuous overstressed zones will exist and to provide basic input to the dynamic analysis on the stress-deformation condition prior to the application of seismic loading.

The ability of the dam to resist seismic forces was analyzed using a rigorous dynamic finite element program which incorporates strain dependent shear modulus and damping parameters. The results were compared with the simplified earthquake induced permanent displacement approach and the pseudo-static limit equilibrium method.

C.2 Static Limit Equilibrium Method

The slope stability analyses were done using the "PC-STABL4" computer program for the general solution of slope stability problems by a two-dimensional limit equilibrium method. In the program the factor of safety against instability of a slope is calculated by an adaptation of both the Modified Bishop and the Modified Janbu method of slices which allows the analysis of trial sliding surfaces of circular as well as non-circular shape. The results were checked using the Morgenstern and Price (1965), and Sarma (1973) methods.

The stability analyses have been performed considering circular sliding planes entirely within the dam as well as composite sliding surfaces extending into and through seams in the foundation bedrock. The analyses generally indicated that the minimum factors of safety occur when the failure surface extends into and through the seams (Wilson and Marsal (1979), Fredlund and Krahn (1977), Fell et al (1987), PFRA (1980), BCHPA (1971), U.S. Corps of Engineers (1970), Janbu (1973), Janbu and Grande (1976), Bishop (1955), Nonveiller (1965), Morgenstern and Price (1965), Eigenbrod and Morgenstern (1972), U.S. Corps of Engineers (1982), Hamel and Spencer (1984), Binnie et al (1967)).

The stability analyses of the dam were carried out using the effective strength parameters listed on Plate C-2. The unit weight of the materials has been selected using the laboratory measured densities and specific gravity as a basis. The porewater pressure ratios, r_u, for the condition at the end of dam construction were selected from data from laboratory tests on samples of soil and rock taken during the site investigation and from the records of porewater pressure response measured in existing dams constructed of similar materials (Wilson and Marsal (1979), Bishop (1952), U.S. Corps of Engineers (1970), Hilf (1948), Bishop (1957), Li (1959), Gibson (1958), Skempton (1954), Bishop (1954), Bishop (1954), Bishop and Henkel (1972), Bishop and Bjerrum (1960), Blight (1970), Gray (1972), Sherman and Clough (1968), Eisenstein and Law (2977), Narita et al (1983)).

Foundation geology and strength are discussed in the Preliminary Engineering Report (UMA/Acres (1986)), and in Appendix B. The foundation bedrock model shown on Plate C-2 was used in the dam analyses. The rationale used to select the seams that have been introduced into the stability analyses has been described in Appendix B.

The model assumed as a basis for design includes the predominantly claystone unit at the base of the Upper Mudrock sequence that exists in varying thicknesses between elevations 1040 m and 1048 m. This is shown as material "F" on Plate C-2. The claystone unit is assumed to include seam V16 (and seam V17) described in Appendix B. The material properties of seam V16 are shown on Plate C-2 as material "H". Seam V12 in Structural Unit No.1 within the Basal Sandstone below elevation 1040 m, is simulated as material "I". The seams are taken to be continuous over the full length of each sliding surface. The seams were introduced into the analysis with a dip of 0.6° in the upstream direction.

The stability analyses show that the minimum factor of safety always occurs along sliding surfaces passing through the highest design seam. Even though a sliding surface passing through seam V12 mobilizes a lower effective angle of shearing resistance, the safety factor is either the same or higher due to the additional normal load and to the increased cross-bedding effective angle of shear resistance at each end of the sliding surface. However the stability of the dam has been assessed on the basis of the lower seam V12 because there is a remote possibility that it could be continuous whereas the upper seam V16 and its parent mudrock are not present over the entire base of the dam.

The results of the stability analyses carried out on the dam, and the assumed critical sliding surfaces are shown in the table on Plate C-2.

All the sliding surfaces into the Lower Mudrock sequence below the Basal Sandstone, must pass through appoximately 37 m of strong standstone with a high cross-bedding angle of shearing resistance, $\phi' = 60^{o}$.

A sensitivity analysis was carried out for a sliding surface passing through the lower Mudrock sequence just below the Basal Sandstone at elevation 1004 m. In this analysis the effective angle of shearing resistance along the sliding surface and the cross-bedding effective angle of shearing resistance in the Basal Sandstone were varied. The results of this analysis is shown on Plate C.3. The analysis shows that, for a sliding surface located just below the Basal Sandstone to be significant for the stability of the dam, it must have an effective angle of shearing reistance equal to or less than 3^{o}. The residual angle of shearing resistance measured and samples taken from the rocks in this region is 12.5^{o}.

The effect of the magnitude of the effective angles of shearing resistance of the design seams on the stability of the dam has been examined with sensitivity analyses which are shown on Plates C-3 to C-10 inclusive.

Stability Cases 1 and 4 are normal operating conditions. The lower required minimum safety factor for Case 4 recognizes that it is a transient condition and that the actual safety factor depends on the amount of porewater pressure dissipation that will take place during the time it will take for the reservoir level to drop. In the analyses it is assumed that the reservoir is drawndown instantaneously and that drainage will only take place in the pervious sand and gravel layer.

All other stability cases are abnormal conditions and lower minimum safety factors than the 1.5 required for the long-term steady state seepage condition are appropriate.

Stability Cases 8 to 11 inclusive are conditions that will exist during construction when parts of the dam will be built to function as upstream cofferdams. The appropriate minimum required safety factors have been set to account for the these temporary conditions during the construction period.

The safety factors computed are either equal to or higher than the required minimum values for all cases of stability conditions analyzed. The two dimensional analyses do not take into account the stabilizing effect of the narrow valley, which will inhibit the development of sliding surfaces of large lateral extent parallel to the dam axis. The effect of the third dimension of any sliding surface, which will be to increase the safety factor, has not been considered.

The stability of the downstream slope of the dam is assumed to be controlled by seam V12. It can be seen that in all cases, the safety factor is either equal or close to that calculated for sliding surfaces that pass through the underlying seam V12. The table on Plate C-2 also shows in brackets the minimum safety factors that were calculated for a sliding surface, passing through seam V16 in the claystone overlying the Basal Sandstone.

The stability of the downstream slope of the dam is governed by Case 1, steady state seepage. In this stability case, the factor of safety will be equal to the required minimum value of 1.5. Case 3 was investigated to determine the safety factor against shear failure if the porewater pressures in the foundation seams do not dissipate as fast as those in the dam core and if some residual construction pore water pressure will still be present after steady state conditions are developed in the core. Three porewater pressure ratios were investigated and it was found that the safety factor will be 1.5 unless some dissipation takes place, in which case the safety factor will be greater. The safety factor will rise to 1.7 if ru

drops to 0.15. In actuality there must be some dissipation of porewater pressures that will occur during the period of a year or more that it will take for steady state seepage conditions to fully develop in the core and in the seam. During this period the safety factor of the downstream slope will be between 1.5 and 1.7 based on the two-dimensional analysis. The actual safety factor will therefore be closer to 1.7 by an indeterminate amount because of the three dimensional constraining effect of the valley walls.

The stability of the upstream slope of the dam is governed by Case 4, rapid reservoir drawdown and by Case 6, at the end of dam construction. In both of these cases, the factor of safety will be equal to the required minimum value of 1.3.

Under a horizontal pseudo-static seismic loading of 0.065 g, both the downstream and the upstream slopes are shown to have a safety factor greater than the required minimum value of 1.0.

If an earthquake occurs at the time that the reservoir is drawndown to minimum supply level, the safety factor of the upstream slope will be unity. This was analyzed for Case 5. This is an extremely unlikely combination of two severe loading cases.

Both Cases 2 and 5 involving earthquake loading were done during the preliminary design of the dam and are listed for the record. The more rigorous dynamic analyses described in Sections C.6.3 largely supersedes the limit equilibrium results.

In Case 7 the stability of the dam during the probable maximum flood is investigated. It is seen that the safety factor against shear failure of both the downstream and upstream slopes is close to those obtained during steady state seepage conditions with the reservoir at

full supply level. The safety factors are well above the required minimum value of 1.2. It is unlikely that steady state seepage conditions could actually develop in relatively impervious core and rockfill during the few days that the reservoir would rise above full supply level.

The stability of the dam was analyzed at the location on the left and right abutments where design seams outcrop at the surface of the dam foundation. The three lowest of these are seams R9, R1, and V18 which lie at elevations 1088.5, 1068.2 referenced to the spillway centreline and 1049.5 m respectively referenced to the dam axis. The calculated safety factors, based on the two dimensional analysis, are less than the required minimums but all are greater than 1.0. The actual safety factor of a sliding mass of fill along a seam in the abutments would be considerably higher than that calculated by the two dimensional analysis and would be greater than the required minimum for the following reasons:

- The seams are horizontal and the dam abutment on which they outcrop is steep and is essentially perpendicular to the dam axis.
- Cross bedding shear strength would have to be mobilized along the face of the wedge of rock that would have to slide out along the seam.
- On the left abutment the axis of the dam is angled in an upstream direction with respect to the face of the abutment.
- On the right abutment the dam arches upstream and forms a wedge in place with the downstream abutment.

The stability of the partially completed portions of the dam while they are acting as cofferdams was analysed in Cases 8 to 11 inclusive. In all cases the calculated safety factor is well above the required minimum.

C.3 Seismic Stability

Past and current practice in assessing the stability of dams under seismic loading is to use the so-called pseudo-static limit equilibrium method (Wilson and Marsal (1979), Williamson and Shaffer (1981), U.A. Corps of Engineers (1977), Sarma (1981), Seed (1982), Matyas et al (1985), TransAlta (1988)). For large dams the method is used for the preliminary shaping of the dam section and the results are then verified by the more rigorous dynamic response analysis (Seed (1966), Seed (1979b), Seed (1982), TransAlta (1988)). In the pseudo-static method the effect of earthquake on a potential sliding mass is represented by an equivalent static horizontal force, determined by the product of the total weight of the sliding mass and a sustained horizontal acceleration which is often assumed to be in the order of 70 percent of the peak maximum credible earthquake horizontal acceleration (Wolofsky and Gough (1985)). However as discussed in the Preliminary Engineering Report, the application of a sustained horizontal acceleration of this magnitude to a sliding mass lying on a seam with low shear strength in a pseudo-static analysis, results in an incorrect indication of how the sliding mass would actually behave under dynamic loading (Seed (1982)). The false conclusion arises because the sustained horizontal pseudo-static force implied in the analysis is unrealistically large as it is derived from the large mass required for stability under static loading. It is considered more realistic to assume a sustained horizontal acceleration of 50 percent of the peak MCE horizontal acceleration for the pseudo-static stability analyses of structures and slopes at the Oldman River dam site.

Assuming the earthquake force acts continuously on the slope material in one direction only, and at the centroid of the mass, the conventional limit equilibrium method of slope stability analysis is applied to evaluate the factor of safety against unacceptably large permanent displacement occurring. A factor of safety of less than unity in the static analysis implies large displacements in the structure, but because of the transient and randomly oscillating

nature of actual earthquake motions, a collapse, in terms of an excessively large displacement, may not occur. Thus, a factor of safety approach does not accurately determine adequate seismic performance.

Based on the seismicity studies described in Wolofsky and Gough (1985), the following earthquake was adopted for design of the Oldman River dam:

- Earthquake Magnitude, M = 6.5,
- Hypocentral Distance, R = 51 km,
- Peak Horizontal Velocity, V_p = 7.5 cm sec $^{-1}$
- Peak Horizontal Acceleration, a_p = 0.13 g
- Sustained Horizontal Acceleration, a_s = 0.065 g

Another approach using the pseudo-static limit equilibrium method to assess the stability of the dam during an earthquake is to calculate the critical acceleration for the dam section which will produce a safety factor of unity and to compare it with the sustained acceleration. The critical acceleration, a_c, is defined as the acceleration applied to the mass which produces a state of limiting equilibrium. Different methods have been used to calculate the critical acceleration for the Oldman River dam. They are listed in Table C-1.

The critical horizontal acceleration calculated for the Oldman River dam using the pseudo-static limit equilibrium approach is approximately equal to 0.065 g. The critical acceleration values given in Table C-1 result in a pseudo-static seismic factor of safety varying between 0.94 and 2.0 for a sustained horizontal acceleration of 0.065g. The safety factor varies considerably with the method of analysis. The low safety factors indicate that it is an unsatisfactory approach for the Oldman River dam analysis.

TABLE C-1

CRITICAL ACCELERATION

Method and Reference	Critical Acceleration, a_c
Limit Equilibrium Method "PCSTABL4"	0.065 g
Newmark (1965)	0.083 g
Sarma (1975)	0.13 g
Sarma and Bhave (1974)	0.09 g
Sarma and Barbosa (1985)	0.061 g
Matyas et al (1985)	0.13 g

C.4 EARTHQUAKE INDUCED PERMANENT DISPLACEMENT APPROACH

Newmark (1965) and Seed (1966) have both criticized the concept of factor of safety as a means of assessing the probable performance of an earth dam during an earthquake. A factor of safety of less than unity in a static analysis is not acceptable as it implies catastrophic failure with large displacements. However, factors of safety of less than unity are acceptable in dynamic analyses, since the earthquake forces act for a short time and alternate in direction, the magnitude of permanent displacements that result may be quite acceptable.

Newmark (1965) indicated that a slope does not necessarily fail just because the dynamic transient stress induced by earthquake motions reaches the strength of the earth material. This conclusion has the important implication that, the stability of a slope during an

earthquake excitation cannot be correctly reflected by the conventional factor of safety through a pseudo-static limit equilibrium analysis. If the factor of safety drops below unity at some time it does not necessarily imply a serious condition. What really matters is the magnitude of the permanent displacement caused by the earthquake when the factor of safety does lie below one.

It is now generally accepted that calculating the magnitude of relative displacement is a more logical and better approach than the factor of safety for assessing the seismic performance of earth dams.

The calculation of the earthquake induced permanent displacement should be carried out using the dynamic finite element method. However, for a dam, if the dynamic response is not significant, the evaluation of the earthquake induced permanent displacement can readily be accomplished by using Newmark's sliding block model, provided that the time history of the ground excitation is known, and soils have very little or no strength loss due to earthquake induced deformations.

The four approximate methods listed in Table C-2 have been used to evaluate the earthquake induced permanent displacement:

Table C-2

EARTHQUAKE INDUCED PERMANENT DISPLACEMENT

Method and Reference	Calculated Permanent Displacements
Newmark (1965)	10.4 mm
Herzog (1985)	20 mm
Lin and Whitman (1986)	10.4 mm
Makdisi and Seed (1978)	28 mm

It is assumed that the soils have very little or no strength loss due to the earthquake induced deformations in all of the methods.

Cyclic triaxial test results on compacted silty clay published by Seed and Chan (1966) and fast direct shear test results on clay published by Skempton (1985) indicated that the assumption of very little or no strength loss under cyclic loading condition is reasonable for the strength of compacted clay till and the residual angle of shearing resistance of the foundation seams. The cyclic laboratory testing performed on samples of compacted till and seams taken during the site investigation has confirmed that there is no strength loss in these materials due to cyclic loadings. The results of the cyclic triaxial testing on compacted till and the cyclic direct shear testing on seams are discussed in Appendix B.

The maximum calculated value of earthquake induced permanent displacement of 28 mm is acceptable for the dam.

C.5 LIQUEFACTION POTENTIAL OF FOUNDATION ALLUVIUM AND COLLUVIUM

The shear stresses developed at any point in a soil deposit during an earthquake appears to be due primarily to the vertical propagation of shear waves in the deposit. This leads to a simplified procedure for evaluating the induced shear stresses (Seed and Idriss (1971)). If the soil column above a soil element at depth h behaved as a rigid body, the maximum shear stress on the soil element would be:

$$(\tau_{max})_r = \frac{\gamma h}{g} a_{max}$$

where a_{max} is the maximum ground surface acceleration and γ is the unit weight of the soil. Because the soil column, behaves as a deformable body, the actual shear stress at depth h, $(\tau_{max})_d$, as determined by a ground response analysis will be less than $(\tau_{max})_r$ and might be expressed by

$$(\tau_{max})_d = r_d \ (\tau_{max})_r$$

Where r_d is a stress reduction coefficient with a value equal to 1 at the ground surface and less than 1 at depth.

The average cyclic stress ratio developed in the field due to the earthquake shaking is given by Seed (1979a) as follows:

$$\frac{(\tau_h)_{avg}}{\sigma'_o} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma'_o} r_d$$

where $(\tau_h)_{avg}$ = average shear stress

σ_o = total overburden pressure on soil layer under consideration

σ'_o = effective overburden pressure on soil layer under consideration

Based on the actual field data, Seed (1979a) developed a correlation between the average cyclic stress ratio causing liquefaction in the field and the modified standard penetration resistance of the sand for the earthquakes of different magnitudes.

Considering that the maximum ground acceleration in the Oldman River Dam Site is 0.13 g, which corresponds to an earthquake with a magnitude equal to 6.5, the critical standard penetration resistance, N_c, calculated from the relationship suggested by Seed (1979a), is equal to 10. With a factor of safety equal to 1.5, suggested by Seed and Idriss (1982), the allowable penetration resistance will be, N = 15. Thus, any sands or silty sands with the N values less than 15 are liquefiable. All such loose deposits will be identified and will be excavated from the foundation of the dam.

The liquefaction studies in China, have also led to a correlation between earthquake shaking conditions causing liquefaction of cyclic mobility and the standard penetration resistance. In this correlation, the critical value of the standard penetration resistance, N_C, separating liquefiable from non-liquefiable conditions is determined by:

$$N_C = \bar{N} (1 + 0.125 (d_s - 3) - 0.05 (d_w - 2)$$

where d_s = depth to sand layer under consideration, m
d_w = depth of water below ground surface, m
$\bar{N}$ = a function of the shaking intensity as follows:

Modified Mercalli Intensity	$\bar{N}$ in blows per foot
VII	6
VIII	10
IX	16

This equation also gives a critical standard penetration resistance, N_C, equal to 10.

C.6 STRESS AND DEFORMATION ANALYSIS

C.6.1 Methodology

Static and dynamic finite element analyses were performed on the Oldman River dam to determine the stability of the structure for static and earthquake dynamic loadings. This is the state-of-the-art approach to the design of such structures and follows the method successfully used by other (Clough and Woodward (1967), Seed et al (1975), Clough and Chopra (1966), U.S. Corps of Engineers (1977), Skermer (1973), Skermer (1975), Seed (1979b), Seed (1982), Khilnani and Byrne (1981), Khilnani et al (1982), Seed (1983), Hirschfeld and Poulos (1973)).

First, a static stress analysis was conducted for the dam cross-section to determine the stress and strain distributions with particular emphasis placed on behavior of the seams in the foundation bedrock. This analysis provided an initial estimate of the static stress distribution and anticipated material deformation characteristics. This provides a more accurate tool to assess the stability of the dam and develops the initial stress distribution to be input to the dynamic stress analysis.

The second phase of the study involved the analysis of dynamic stresses and displacements developed in the dam when subjected to the design earthquake accelerations. The stresses were compared to available material strengths and the displacements compared to allowable maximum deformations to assess the dynamic factor of safety.

Two separate nonlinear finite element models were used in the analysis. The pre-earthquake stress condition was evaluated using the nonlinear finite element program "SAN" developed by M. M. Davachi at the Imperial College, University of London, England (Davachi (1978)). This program is a nonlinear two-dimensional finite element model which uses "gravity turn-on" loading by simulating the construction of the dam in layers utilizing the dense liquid technique. The dynamic stress analysis was conducted using the two-dimensional finite element program "QUAD4" developed by Idriss et al (1973) at the Earthquake Engineering Research Centre, University of California, Berkeley.

C.6.2 Static Finite Element Analysis

C.6.2.1 Model Configuration

The static stress analysis was performed using the "SAN" program. The program utilizes 8 noded iso-parametric elements and nonlinear soil material laws. The nonlinearity arises from

the dependency of material deformation and shear strength properties upon the effective stress state and the effective mean stress. The dam cross-section was modelled by a two dimensional plane strain mesh as shown on Plate C-11.

C.6.2.2 Material Properties

The simplified dam section was defined using seven distinct material types. The shear strength and deformation material laws input to the model were estimated from laboratory triaxial and direct shear tests conducted on compacted and undisturbed samples obtained from the dam site and from data presented in the literature. The seven soil units used to represent the dam section are as follows:

Unit 1 - Compacted Glacial Till Core

The core of the dam consists of compacted silty clay till obtained from borrow areas at the dam site. Consolidated undrained triaxial tests with pore pressure measurements were conducted on compacted samples, they provided estimates of the initial Young's modulus (E_i), Poisson's ratio (ν), effective shear strength parameters (C', ϕ'), and the nonlinear stress-strain relationship. Table C-3 illustrates the values used in static stress analysis.

Unit 2 - Compacted Granular and Random Fill Sheel Material

The material comprising the shells of the dam consists of compacted granular and random fill. Material properties for this unit were selected from results published in the literature for similar materials and are tabulated in Table C-3. The filter zones adjacent to the central core are included within the shell zone because of the similarity of the two materials.

Unit 3 - Foundation Alluvium and Colluvium

The river valley deposits consist of a substantial quantity of colluvium material overlying the alluvial river granular deposits. Complete excavation of the colluvium is planned for areas beneath the dam shells. A layer of this material was included beneath the dam shells for the finite element analysis in order to evaluate the liquefaction risk. The material properties used in the analysis were estimated from values presented in the literature.

Units 4 and 5 - Seams Within Foundation Bedrock

The design seams located in the dam foundation and their shear strengths discussed in Appendix B. These seams are modelled as thin continuous horizontal layers located within the bedrock units.

Units 6 and 7 - Intact Mudrock and Sandstone Units

The upper mudrock unit is modelled as a thin continuous horizontal layer in the foundation. The sandstone unit directly underlies the mudrock layer. Material deformation properties were obtained from laboratory triaxial testing data. The intact bedrock units were modelled as linear-elastic materials since the shear stresses developed from static loading do not approach the available strength of these units.

TABLE C-3

MATERIAL PROPERTIES USED IN STATIC ANALYSIS

MATERIAL	C' (kPa)	ϕ'	γ (kN/m^3)	ν	E_i (MPa)	
1. Compacted Till (Core)	0	25°	19.6	0.4	20 + 67	(σ'_m)
2. Compacted Granular (Shells)	0	35°	21.6	0.3	84 + 267	(σ'_m)
3. Colluvium/Alluvium (Foundation)	0	28°	21.6	0.3	84 + 267	(σ'_m)
4. Seam in Mudrock (Foundation)	0	17°	23.6	0.4	400	
5. Seam in Sandstone (Foundation)	0	12.5°	23.6	0.3	400	
6. Intact Mudrock (Foundation)	-	Linear	23.6	0.4	2,000	
7. Intact Sandstone (Foundation)	-	Linear	23.6	0.3	17,000	

σ'_m = mean effective stress = $(\sigma'_1 + \sigma'_3)/2$

C.6.2.3 Simplifying Assumptions

The following simplifying assumptions and procedures were used in the static analysis:

- The dam cross-section was modelled in two-dimensional plane strain.
- Foundation units were modelled as horizontal continuous layers.
- Initial effective stresses used in the foundation assumed water table at ground surface.

- Foundation intact bedrock units were modelled as linear elastic.
- Embankment construction was simulated by incremental switch-on gravity procedure.
- Very small effective cohesion was added to frictional materials to prevent numerical failure during the first loading increment.
- Only the end-of-construction condition was analysed.

C.6.2.4 Results and Conclusions

The results of the static finite element analysis are presented on Plates C-12 to C-17 These include:

- distribution of major and minor principal effective stresses (σ'_1 , σ'_3).
- distribution of mean effective stress, $\sigma'_m = (\sigma'_1 + \sigma'_3)/2$
- ratio of shear stress to shear strength, $(\sigma_1 - \sigma_3)/(\sigma_1 - \sigma_3)_f$
- deflected profile (from elastic settlement)
- comparison of the stresses along dam axis to the theoretical full overburden pressure and pore pressures caused by hydrostatic head at full supply level.

The following conclusions are forwarded based upon consideration of the static finite element results:

- There is a reasonable degree of stress transfer between the compacted till core and the compacted alluvial shell material arising from the stiffness contrast between these materials. The ratio of vertical stress predicted from the finite element results compared to the theoretical full overburden pressure is equal to 0.65.

- Considering the maximum head in the reservoir from the PMF level the water pressure anywhere in the core is less than the total vertical and mean stresses predicted from the finite element analysis. Thus, the hydraulic fracturing will not take place in the core.
- The shear stresses in a few of the elements in one area representing the seams in the foundation bedrock units were equal to the shear strength. However, this condition was limited to the one area only and there was no development of a continuous band of such elements.

C.6.3 Dynamic Response Analysis

C.6.3.1 General

The dynamic stress analysis was conducted using the nonlinear finite element program "QUAD4". The program incorporates variable damping quadrilateral elements with nonlinearity resulting from the variation in shear modulus (G) and damping ratio (λ) and shear strain (γ). The solution procedure is based on an overrelaxation iterative procedure which solves for final values of shear modulus and damping ratio which are compatible with the average strain developed in each element. Output from the program includes resulting maximum and final displacements, maximum dynamic shear stresses, maximum accelerations, and final shear strains with compatible shear modulus and damping ratios.

Material laws input to the program were developed from preliminary laboratory testing results and from procedures outlined by Seed and Idriss (1970). Initial static stresses were input to the analysis in the form of initial values of shear modulus (G_{max}) developed as a function of the initial effective stresses (from static stress analysis).

The dynamic analysis requires a time history of acceleration. Since no local earthquake records for the dam site are available, the time history of acceleration recorded at Taft during the 1952 earthquake that occurred in Kern County, California was used. The earthquake acceleration was normalized to a peak horizontal acceleration of 0.13 g. This earthquake is characterized by a period of approximately 0.3 seconds and has a duration of 30 seconds. Strong motions occur from time t = 5 seconds until time t = 15 seconds. The acceleration-time plot of the earthquake used in the analysis is shown on Plate C-18.

C.6.3.2 Model Configuration and Material Properties

The mesh used in the static analysis was duplicated for the dynamic study to facilitate easy superposition of the two model results. The "QUAD4" program utilizes 4 noded quadrilateral elements.

A total of seven material units was again used to define dam cross-section. Material properties were input in the form of initial shear modulus for small strains (G max) and Poisson's ratio (ν). The program incorporates a material algorithm which modifies the shear modulus and damping ratio for each element to be compatible with the shear strain developed within the element in response to the earthquake load.

Table C-4 lists the initial shear modulus (G) and the Poisson's ratio (ν) values input to the model as function of initial effective confining pressure:

$$\sigma'_m = \frac{(1 + 2 K_o)\sigma'_v}{3} \approx \frac{\sigma'_1 + \sigma'_3}{2}$$

With the exception of the bedrock units, all materials were assigned initial shear modulus values based upon the recommendations of Seed and Idriss (1970) since no site specific laboratory cyclic test data was available at the time of the study. Initial shear modulus was developed from the following expression:

$$G_{max} = 1000\ (K_2)_{max}\ (\bar{\sigma}_m)^{1/2}$$

where:

G_{max} = initial shear modulus corresponding to small strains (KSF)

$(K_2)_{max}$ = dimensionless factor depending upon material characteristics (Seed and Idriss (1970))

$\bar{\sigma}_m$ = effective mean principal stress (from static analysis), (PSF)

The material properties for the bedrock units were estimated from static triaxial and direct shear box laboratory tests.

C.6.3.3 Assumptions Used in Dynamic Analysis

The following assumptions were used in the dynamic modelling:

- Initial shear modulus values for all materials (except bedrock units) were estimated from literature (Seed and Idriss (1970)). Bedrock units were estimated using static laboratory testing and geophysical survey results and later confirmed by the dynamic laboratory test described in Appendix B.
- The shear modulus used in foundation bedrock was not varied with effective confining pressure.

TABLE C-4

INITIAL MATERIAL PROPERTIES INPUT TO DYNAMIC ANALYSIS

MATERIAL	γ (PCF)	K_2	CONSTANT G_{max} (KSF)	ν
1. Compacted Till (Core)	125	50	—	0.4
2. Compacted Granular (Shells)	137	75	—	0.3
3. Colluvium/Alluvium (Foundation)	137	40	—	0.3
4. Seam in Mudrock (Foundation)	150	50	—	0.4
5. Seam in Sandstone (Foundation)	150	50	—	0.4
6. Intact Mudrock (Foundation)	150	—	13,900	0.4
7. Intact Sandstone (Foundation)	150	—	118,000	0.3

- The dam cross-section was modelled in two dimensional plane strain.
- The foundation units were assumed to be horizontal and continuous.
- The dam was represented by seven different material types for embankment and foundation.
- The shear modulus and damping ratios of all materials were modified by built-in material laws to produce shear-strain compatible results.
- Boundary conditions on foundation allowed no vertical movements to simulate the semi-infinite continuum.

- Seams in foundation bedrock units were assigned same shear modulus values as the dam core.
- The analysis was conducted using the Taft earthquake normalized to a maximum horizontal acceleration of 0.13 g. No vertical acceleration was considered.
- It was assumed that there are no losses in the strength of compacted till and in the residual angle of shearing resistance of the seams under dynamic earthquake loading. This was later confirmed by the cyclic laboratory testing described in Appendix B.

C.6.3.4 Results and Conclusions

The results of the dynamic analyses are illustrated in Plates C-19 to C-25. These include:

- combined static and maximum dynamic principal stress distributions.
- ratio of combined static and dynamic stresses to available static shear strength.
- maximum horizontal accelerations.
- ratio of maximum dynamic shear stress to combined static and dynamic mean stress.
- maximum and final resultant displacements.

The following conclusions are drawn from the results of the dynamic analyses:

- The analysis indicated a high sensitivity to the period of the input earthquake excitation. As the dominant period of the earthquake approached the natural period of the structure (approximately 1.2 seconds) resonance occurred and amplification of earthquake accelerations, displacements and stresses resulted.

- When the static and dynamic stresses are combined, the analysis indicates that localized band of elements will exist in the foundation seams along which the shear stress is equal to the static shear strength. This does not represent failure of the dam as the bands of overstressed elements are localized and limited to the foundation and do not extend through the dam fill and under such alternating, short period forces the corresponding magnitude of permanent displacement would be small.
- It is of interest to note that the dynamic shear stresses generated by earthquake acceleration were always additive on one side of the dam and cancelling on the other side.
- The maximum displacement during earthquake was equal to 57 mm, and maximum final permanent displacement was calculated to be 21 mm.
- The maximum horizontal acceleration at crest was calculated to be to 0.32 g.
- The period of the structure varies significantly when subjected to earthquakes having differing dominant periods. In the dam, larger period earthquakes produce larger shear strains (γ). This in turn reduces the shear modulus (G) and increases the damping ratio (λ) producing longer period for the structure.

C.7 STABILITY ANALYSIS FOR DYKE

A typical cross-section of the dyke is shown on Plate C-26.

The slope stability analysis for the dyke has been carried out using the "PCSTABL4" computer program for the general solution of slope stability problems by a two dimensional limiting equilibrium method. The Modified Bishop and Modified Janbu method of slices were used to determine the factor of safety against instability.

In the analyses, circular failure planes extending through the dyke fill and the foundation soils were assumed.

The effective strength parameters used in the analyses are given in Table C-5.

Only the pseudo-static limit equilibrium method was used to assess the stability of the dyke under seismic loading.

The critical potential failure surfaces are shown on Plate C-27. The results of the stability analyses are summarized in Table C-6.

TABLE C-5
MATERIAL PROPERTIES USED IN LIMIT EQUILIBRIUM ANALYSIS FOR DYKE

	Effective Cohesion C' (kPa)	Effective Angle of Shearing Resistance ϕ'	γ Unit Weight (kN/m^3)
Foundation Materials			
Glaciolacustrine Clay	5.0	22°	21.6
Glacial Till	15.0	25°	21.6
Compacted Materials			
Zone 1 A	5.0	22°	21.6

TABLE C-6

DYKE STABILITY ANALYSIS

Operating Conditions	Required Minimum Factor of Safety	Calculated Factor of Safety			
		Upstream Slope		Downstream Slope	
		Safety Factor	Sliding Surface	Safety Factor	Sliding Surface
1. Full Supply Level with Steady State Seepage	1.5	1.7	1	1.7	5
2. Rapid Drawdown from Full Supply Level	1.3	1.6	2	---	-
3. Full Supply Level with Steady State Seepage and Horizontal Pseudostatic Seismic Loading 0.065 g.	1.0	1.3	1	1.3	5
4. Full Supply Level with Steady State Seepage & Undissipated Foundation Pore Pressure, $r_u = 0.3$	1.3	1.8	1	1.7	8
5. As Above with $r_u = 0.4$	1.3	1.6	4	1.5	7
6. End of Construction Pore Pressure in Dyke and Foundation $r_u = 0.3$	1.3	1.5	1	1.5	9
7. As Above with $r_u = 0.4$	1.3	1.3	1	1.3	9
8. Maximum Flood Level with Steady State Seepage	1.2	2.1	4	1.7	6
9. Rapid Drawdown from Full Supply Level with Horizontal Pseudostatic Seismic Loading 0.065 g.	1.0	1.2	3	---	-

C.8 SUMMARY AND CONCLUSIONS

C.8.1 Dam

Limit equilibrium and finite element analyses have been carried out to determine the stability of the Oldman River dam under static and earthquake induced dynamic loads.

The limit equilibrium method was used to perform static and pseudo-static slope stability analyses and to establish the overall shape of the dam. The bedrock seams in the dam foundation were modelled using the shear strength parameters described in Appendix B. The static safety factors computed from the static limit equilibrium analyses are either equal to or greater than the required minimum values.

The pseudo-static limit equilibrium stability analyses indicated that the safety factors under sustained seismic loading of 50 percent of the peak MCE horizontal acceleration are approximately equal to unity.

A factor of safety of less than unity in a static analysis is not acceptable as it implies catastrophic failure with large displacements. However, factors of safety of less than unity are acceptable during dynamic loading, since the earthquake forces act for a short time and alternate in direction, and the magnitude of permanent displacement is the criterion of acceptability. If only small displacements occur, they may be quite acceptable (Newmark (1965)).

The finite element method was used to carry out static and dynamic stress and deformation analyses of the dam for static and seismic loads.

The results of static finite element analyses indicated that the hydraulic fracturing will not take place in the core of the dam.

The shear stresses in a few of the elements in one area representing the seams in the foundation bedrock units were equal to the shear strength after imposing the pre-earthquake stresses. However this condition was limited to the one area only and there was no development of a continuous band of such elements.

The time history of acceleration developed at Taft during the 1952 Kern County, California earthquake normalized to peak horizontal acceleration of 0.13 g. was used in the dynamic analysis.

Deformation parameters and damping factors used in the dynamic response analyses were initially taken from literature. It was assumed that there are no losses in the strength of compacted till and in the residual angle of shearing resistance of the seams under dynamic earthquake loading. The site-specific cyclic laboratory testing subsequently performed has confirmed that these assumed values are realistic. The results of the cyclic testing and compacted till are presented in Appendix B.

When the static and dynamic stresses are combined, the analysis indicates that localized band of elements will exist in the foundation seams along which the shear stress is equal to the static shear strength. This does not represent failure of the dam as the bands of overstressed elements are limited to the foundation and do not extend through the dam fill and under such alternating, short period forces the corresponding magnitude of permanent displacement would be small.

Dynamic response analyses showed that the maximum displacement during earthquake was equal to 57 mm and maximum final plastic displacement was equal to 21 mm. These are considered to be acceptable.

The evaluation of liquefaction potential of foundation alluvium and colluvium indicated that any sands or silty sands with N values less than 15 (corresponding to a relative density of 50 percent) are liquefiable and they will be identified and excavated from the foundation of the dam.

C.8.2 Dyke

The factor of safety against instability computed from the limit equilibrium analyses are either equal to or greater than the required minimum values.

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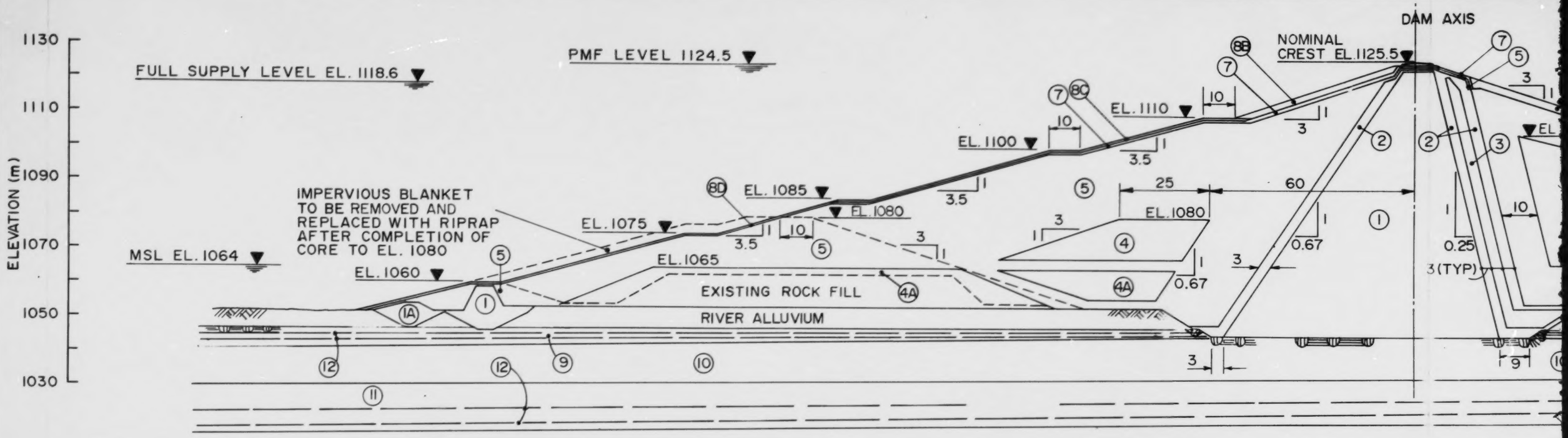
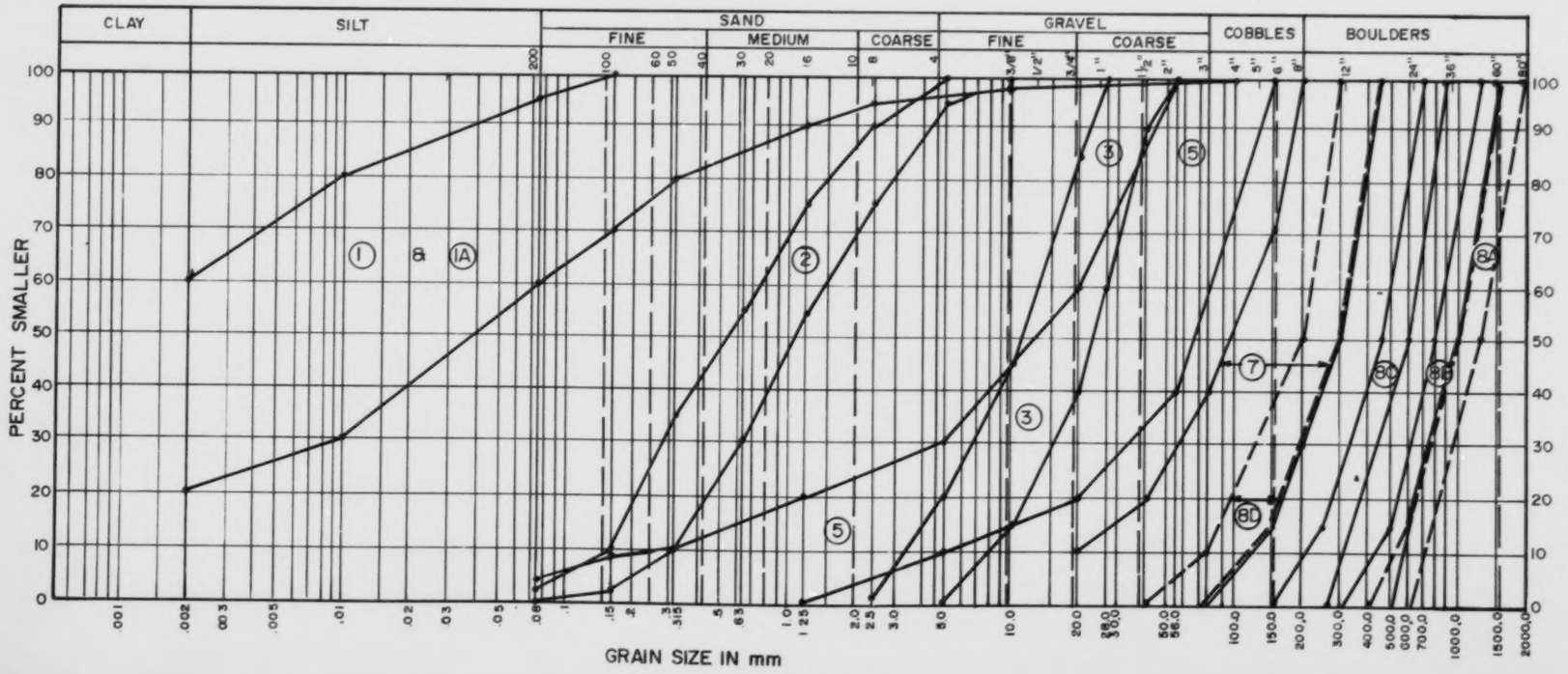
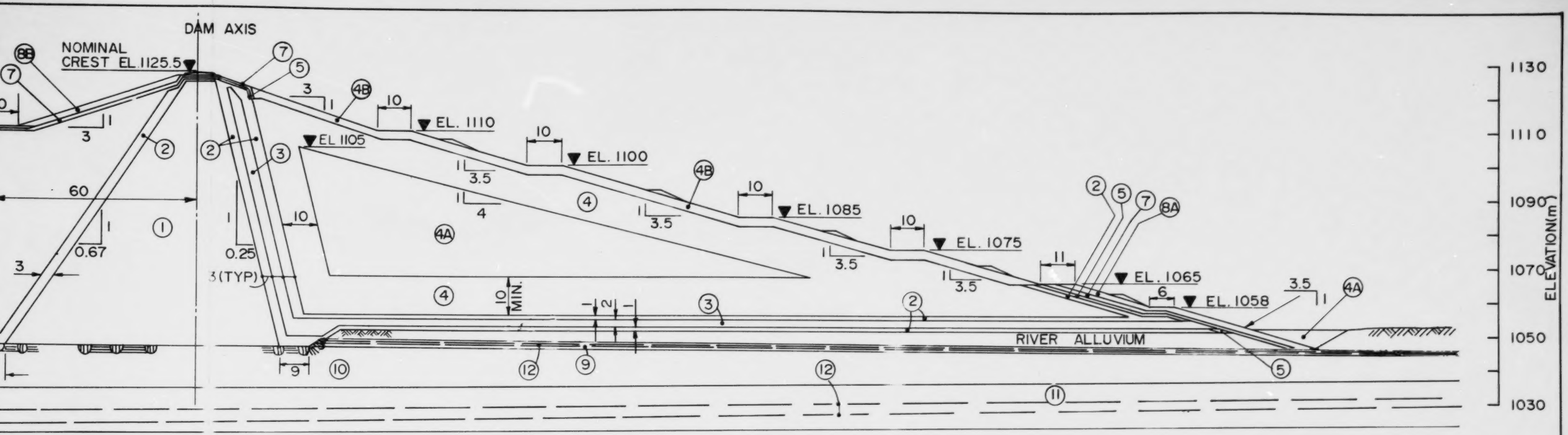




LEGEND:

① CORE
⑯ IMPERVIO
② FILTER
③ DRAIN
④ ROCKFIL
④A UNCLASS
④B OVERSIZ
⑤ GRANULA
⑥ PERMAN
⑦ RIPRAP



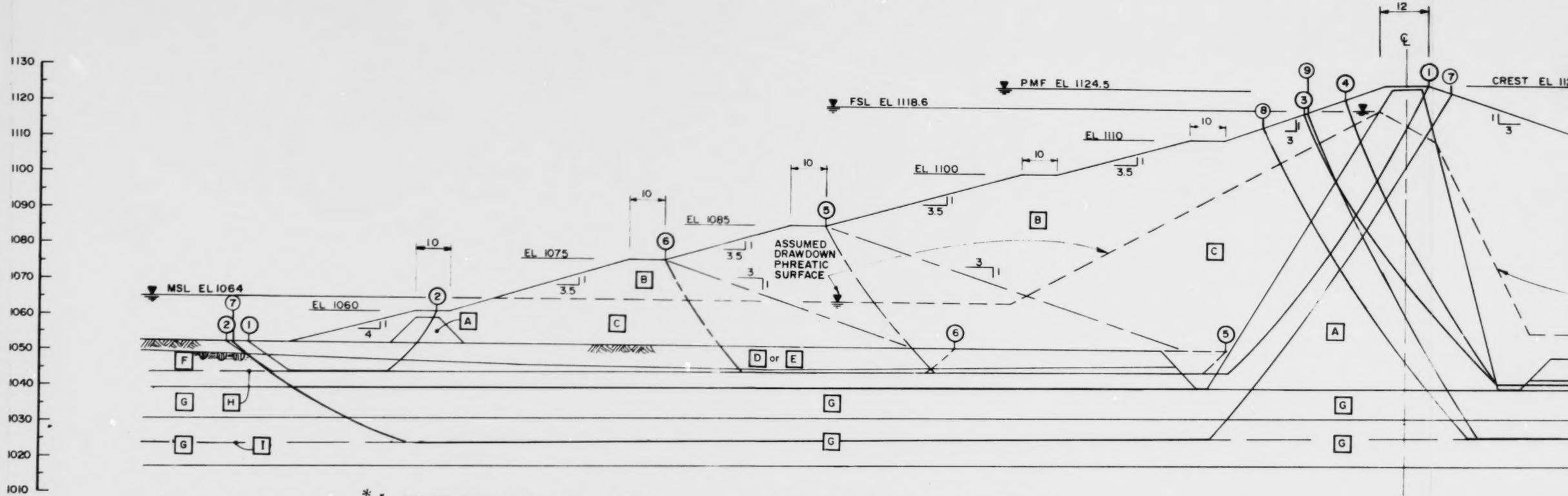
LEGEND

(1) CORE
(1A) IMPERVIOUS FILL
(2) FILTER
(3) DRAIN
(4) ROCKFILL
(4A) UNCLASSIFIED FILL
(4B) OVERSIZE ROCKFILL
(5) GRANULAR FILL
(6) PERMANENT ROAD GRAVEL
(7) RIPRAP BEDDING
(8A) RIPRAP CLASS A
(8B) RIPRAP CLASS B
(8C) RIPRAP CLASS C
(8D) RIPRAP CLASS D
(9) CLAYSTONE
(10) SANDSTONE
(11) CLAYSTONE/SANDSTONE INTERBEDS (STRUCTURAL UNITS 1 & 2)
(12) SEAMS

Alberta
OLDMAN RIVER DAM

DAM SECTION

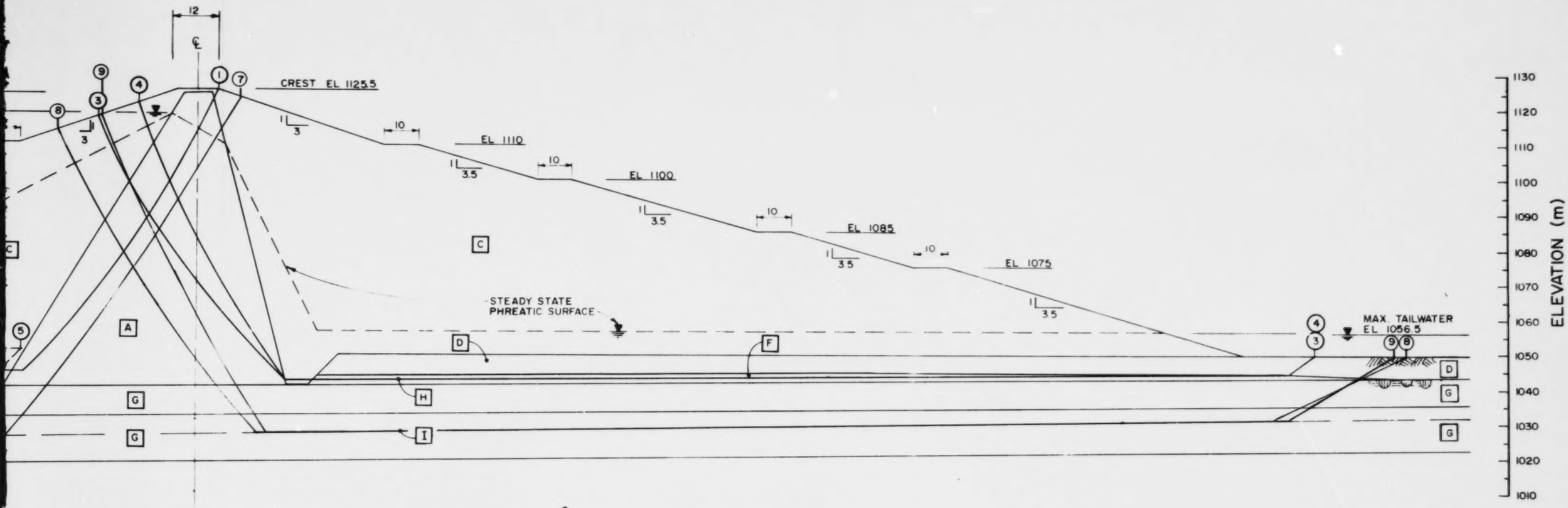
PLATE C-1



* r_u VALUES PERTINFNT TO FOUNDATION SEAMS ONLY.

NOTE: FIGURES SHOWN IN BRACKETS ARE FOR SLIDING SURFACES PASSING THROUGH MATERIAL H

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQ'D.	UPSTREAM F.S.	UPSTREAM FAILURE SURFACE	DOWNSTREAM F.S.	DOWNSTREAM FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN THE DAM AND FOUNDATION.	1.5	1.9 (1.9)	7 (1)	1.5 (1.5)	8 (3)
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN THE DAM AND FOUNDATION PLUS PSEUDO-STATIC SEISMIC HORIZONTAL LOADING 0.065g.	1.0	1.3 (1.2)	7 (1)	1.1 (1.1)	8 (3)
3. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN DAM AND CONSTRUCTION PORE PRESSURES IN FOUNDATION.	1.3	-	-	*r_u = 0.3 1.5 (1.3) *r_u = 0.2 1.6 (1.4) *r_u = 0.15 1.7 (1.5)	8 (3)
4. RAPID RESERVOIR DRAW-DOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL.	1.3	1.3 (1.3)	7 (1)	-	-
5. RAPID RESERVOIR DRAW-DOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL PLUS PSUEDO-STATIC SEISMIC HORIZONTAL LOADING 0.065g.	1.0	1.0 (1.0)	7 (1)	-	-
6. END OF CONSTRUCTION PORE PRESSURES IN DAM AND FOUNDATION BEFORE RESERVOIR IMPOUNDING.	1.3	1.4 (1.3)	7 (1)	1.5 (1.3)	9 (4)
7. RESERVOIR AT PROBABLE MAXIMUM FLOOD LEVEL WITH STEADY STATE SEEPAGE CONDITIONS IN THE DAM AND FOUNDATION.	1.2	2.0 (1.9)	7 (1)	1.4 (1.4)	8 (3)
8. COFFERDAM CREST ELEVATION 1,085 m. RIVER LEVEL ELEVATION 1,083. WITH STEADY STATE SEEPAGE CONDITIONS IN COFFERDAM AND FOUNDATION.	1.4	2.1	2	1.6	5
9. COFFERDAM CREST ELEVATION 1,085. END OF CONSTRUCTION PORE PRESSURES IN COFFERDAM AND FOUNDATION.	1.2	1.4	2	1.3	5
10. COFFERDAM CREST ELEVATION 1,075, RI LEVEL ELEVATION 1,073, WITH STEADY STATE SEEPAGE CONDITIONS IN DAM AND FOUNDATION.					
11. COFFERDAM CREST ELEVATION 1,075 AND CONSTRUCTION PORE PRESSURES IN DAM AND FOUNDATION.					



UPSTREAM		DOWNSTREAM		FACTOR OF SAFETY / CASE	MIN. F.S. REQ'D.	UPSTREAM		DOWNSTREAM	
F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE			F.S	FAILURE SURFACE	F.S	FAILURE SURFACE
2.0	7	1.4	8	10. COFFERDAM CREST ELEVATION 1,075, RIVER LEVEL ELEVATION 1,073, WITH STEADY STATE SEEPAGE CONDITIONS IN DAM AND FOUNDATION.	1.4	2.1	2	1.6	6
(1.9)	(1)	(1.4)	(3)						
2.1	2	1.6	5	11. COFFERDAM CREST ELEVATION 1,075 AND CONSTRUCTION PORE PRESSURES IN DAM AND FOUNDATION.	1.2	1.4	2	1.3	6
.4	2	1.3	5						

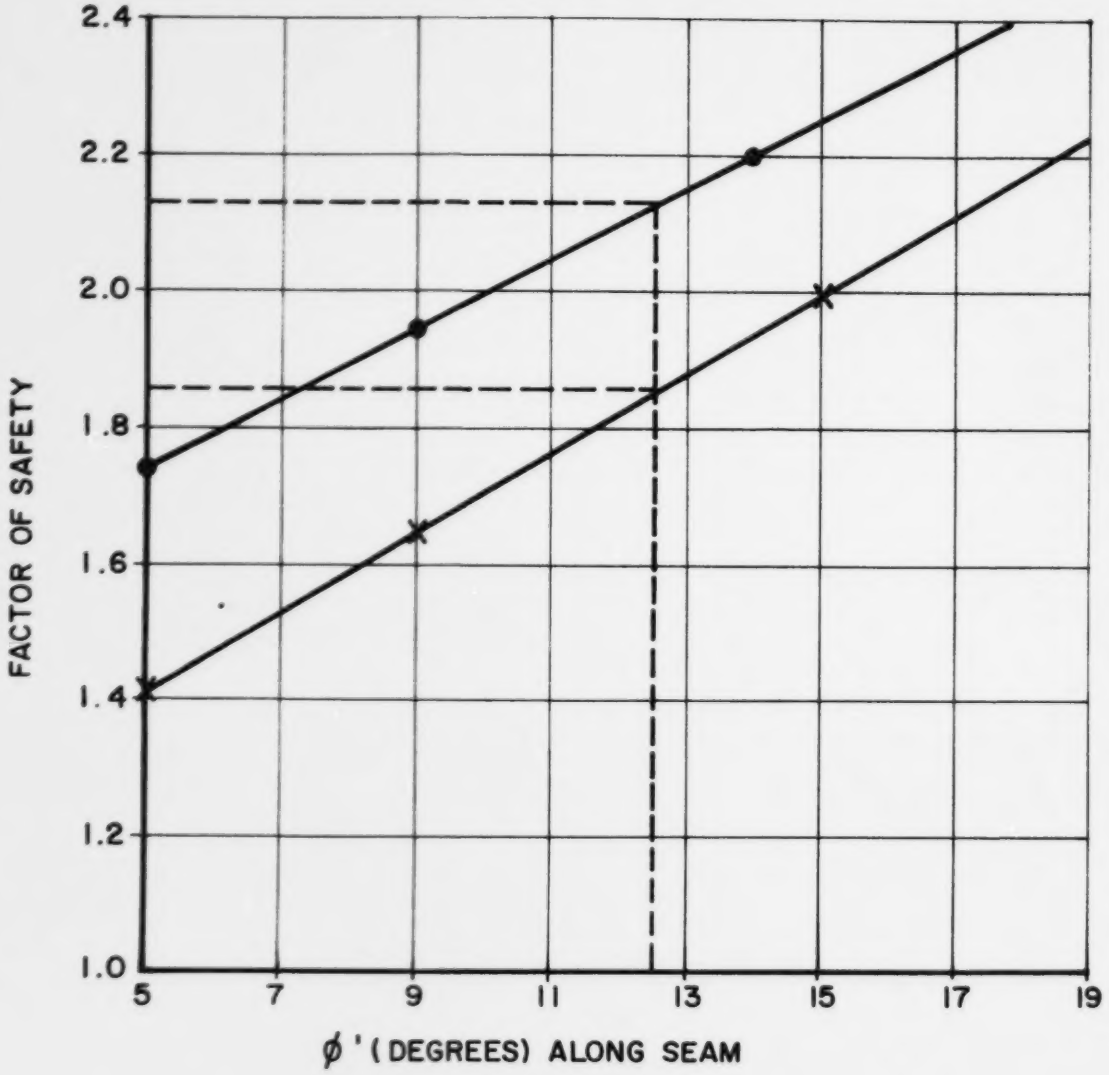
MATERIAL	UNIT WEIGHT γ (kN/m³)	EFFECTIVE COHESION C'	ANGLE OF EFFECTIVE SHEARING RESISTANCE ϕ'	END-OF CONSTRUCTION POREWATER PRESSURE RATIO r_u
A - COMPACTED CORE	21.6	0	25°	0.4
B - GRANULAR FILL	21.6	0	35°	0
C - ROCK FILL	21.6	0	35°	0.1
D - RIVER ALLUVIUM	21.6	0	35°	0.1
E - RIVER COLLUVIUM	21.6	0	25°	0.3
F - CLAYSTONE (CROSS BEDDING)	23.6	0	35°	0.3
G - SANDSTONE (CROSS BEDDING)	23.6	0	60°	0.2
H - SEAM IN CLAYSTONE	23.6	0	17.0°	0.3
I - SEAM IN MUDROCK IN BASAL SANDSTONE	23.6	0	12.5°	0.3

0 10 20 30 m

Alberta WRC III
OLDMAN RIVER DAM

DAM STABILITY ANALYSIS

PLATE C-2



DAM - DOWNSTREAM SLOPE - STEADY STATE SEEPAGE
SLIDING THROUGH SEAM No. V8, EL = 1004 m

LEGEND

CROSS BEDDING SHEAR STRENGTH
IN BASAL SANDSTONE

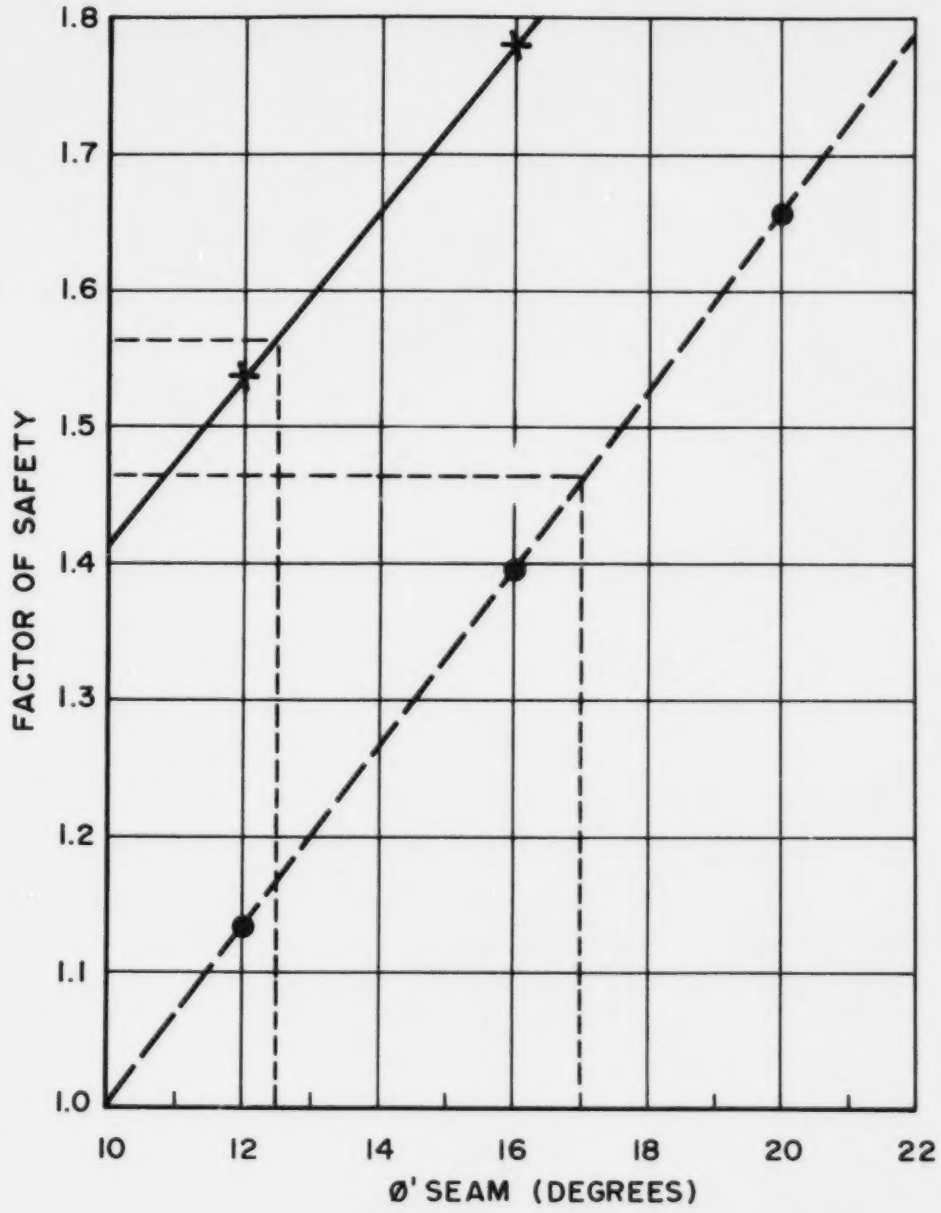
● ϕ_x' = 50° , C_x' = 0

X ϕ_x' = 40° , C_x' = 0

Alberta Power
OLDMAN RIVER DAM

FACTOR OF SAFETY VS ϕ'
ALONG SEAM V8

PLATE C-3



DAM - DOWNSTREAM SLOPE

STEADY STATE
SEEPAGE CONDITION

SLIDING THROUGH SEAM AT :

● - UPPER CLAYSTONE OVERLYING BASAL SANDSTONE-SEAM V16

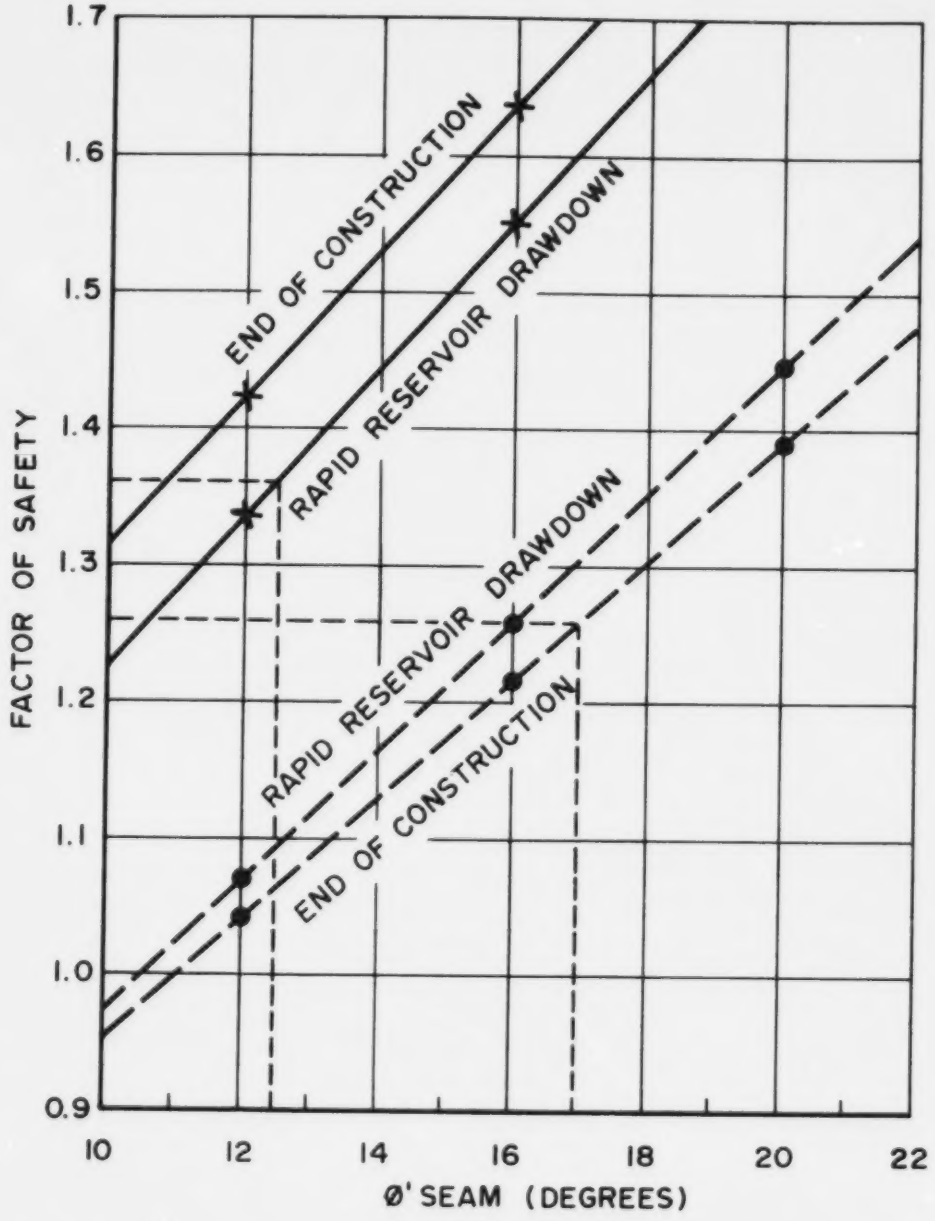
X - CLAYSTONE IN BASAL SANDSTONE - SEAM V12

NOTE
SEAM V16 IS NOT CONTINUOUS IN DAM FOUNDATION

Atcoma Limited
OLDMAN RIVER DAM

FACTOR OF SAFETY VS Ø'
DAM-DOWNSTREAM SLOPE

PLATE C-4



DAM - UPSTREAM SLOPE

END OF CONSTRUCTION AND RAPID RESERVOIR DRAWDOWN

SLIDING THROUGH SEAM AT :

● - UPPER CLAYSTONE OVERLYING BASAL SANDSTONE - SEAM V16

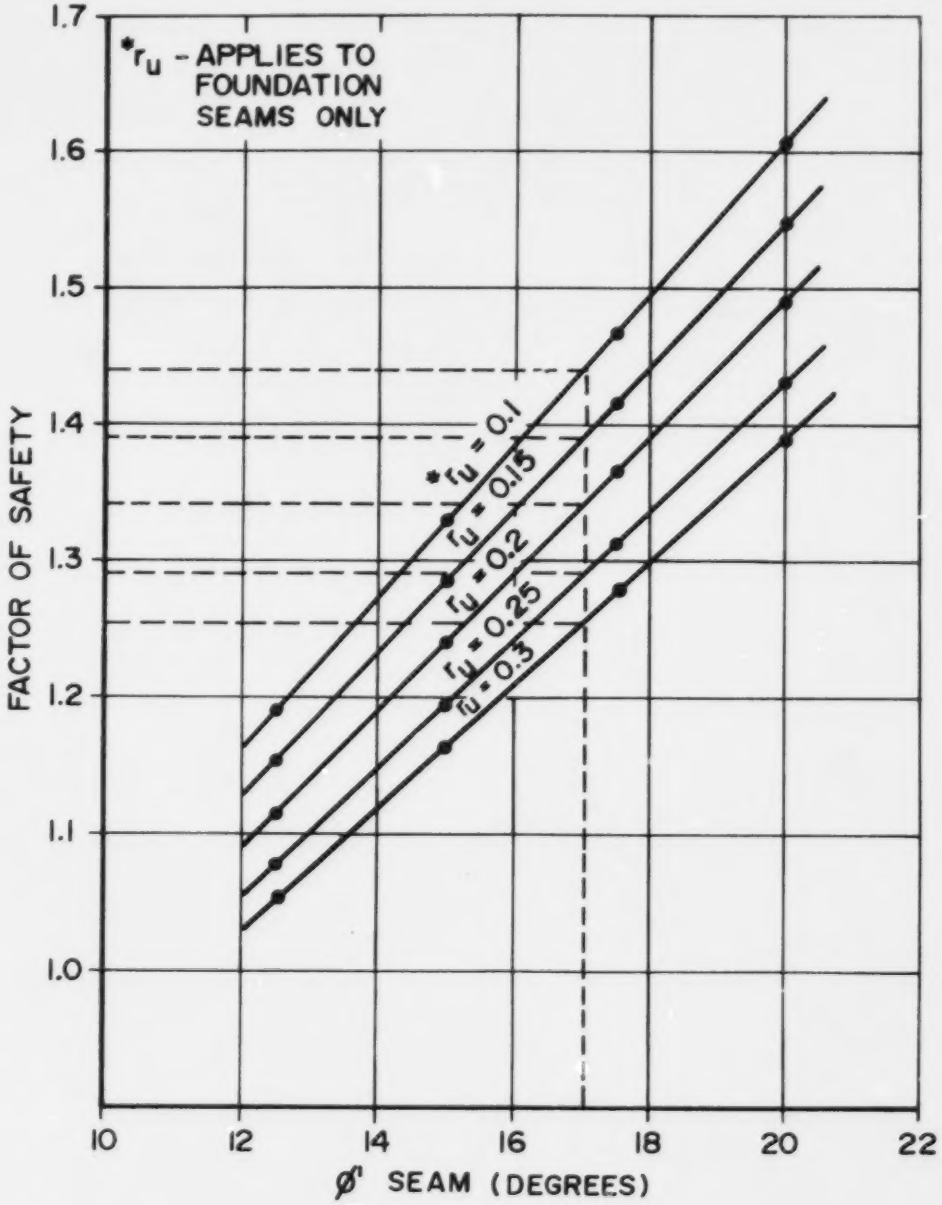
X - CLAYSTONE IN BASAL SANDSTONE - SEAM V12

NOTE
SEAM V16 IS NOT CONTINUOUS IN DAM FOUNDATION

Alcoa Minerals
OLDMAN RIVER DAM

FACTOR OF SAFETY VS Ø'
DAM - UPSTREAM SLOPE

PLATE C-5



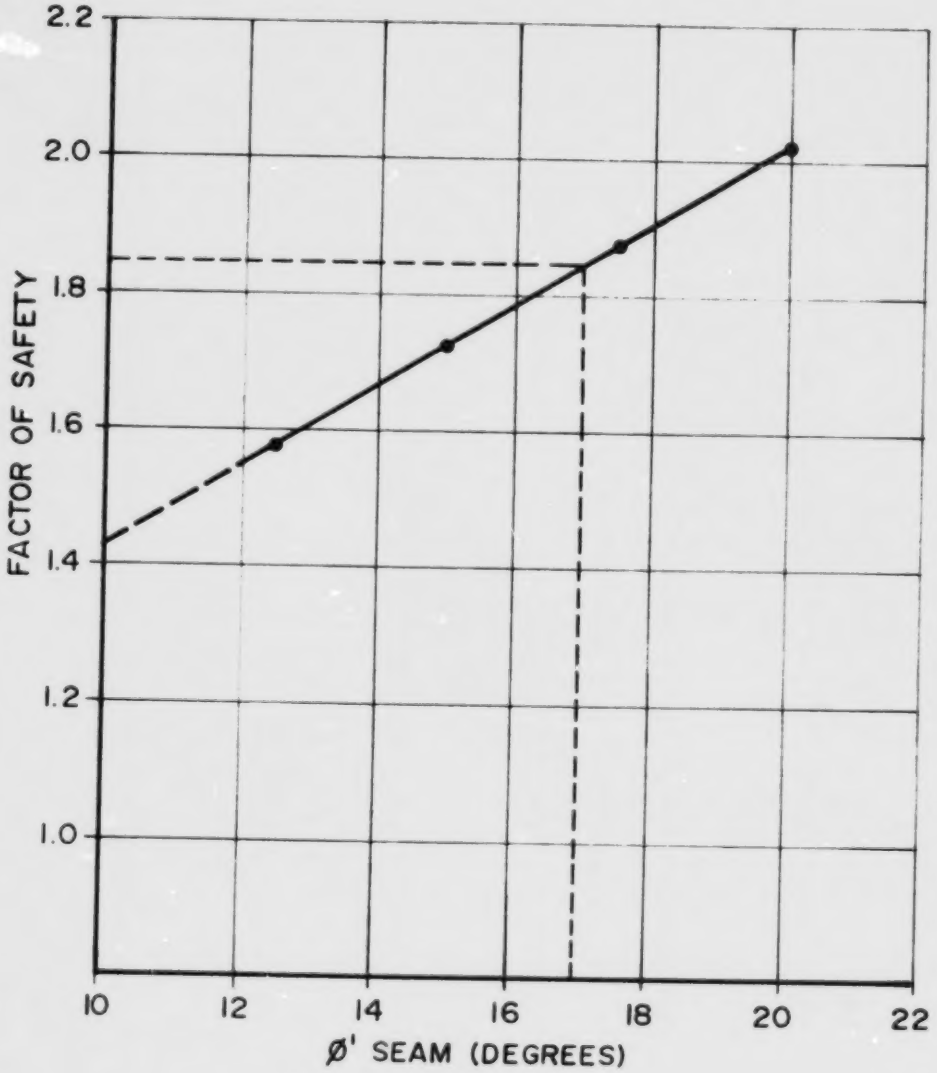
NOTE
SEAM V16 IS NOT CONTINUOUS IN DAM FOUNDATION

DAM - UPSTREAM SLOPE
END OF CONSTRUCTION
SLIDING THROUGH SEAM V16 IN UPPER CLAYSTONE OVERLYING BASAL SANDSTONE

Alberta Power
OLDMAN RIVER DAM

FACTOR OF SAFETY VS ø'
DAM - UPSTREAM SLOPE

PLATE C-6



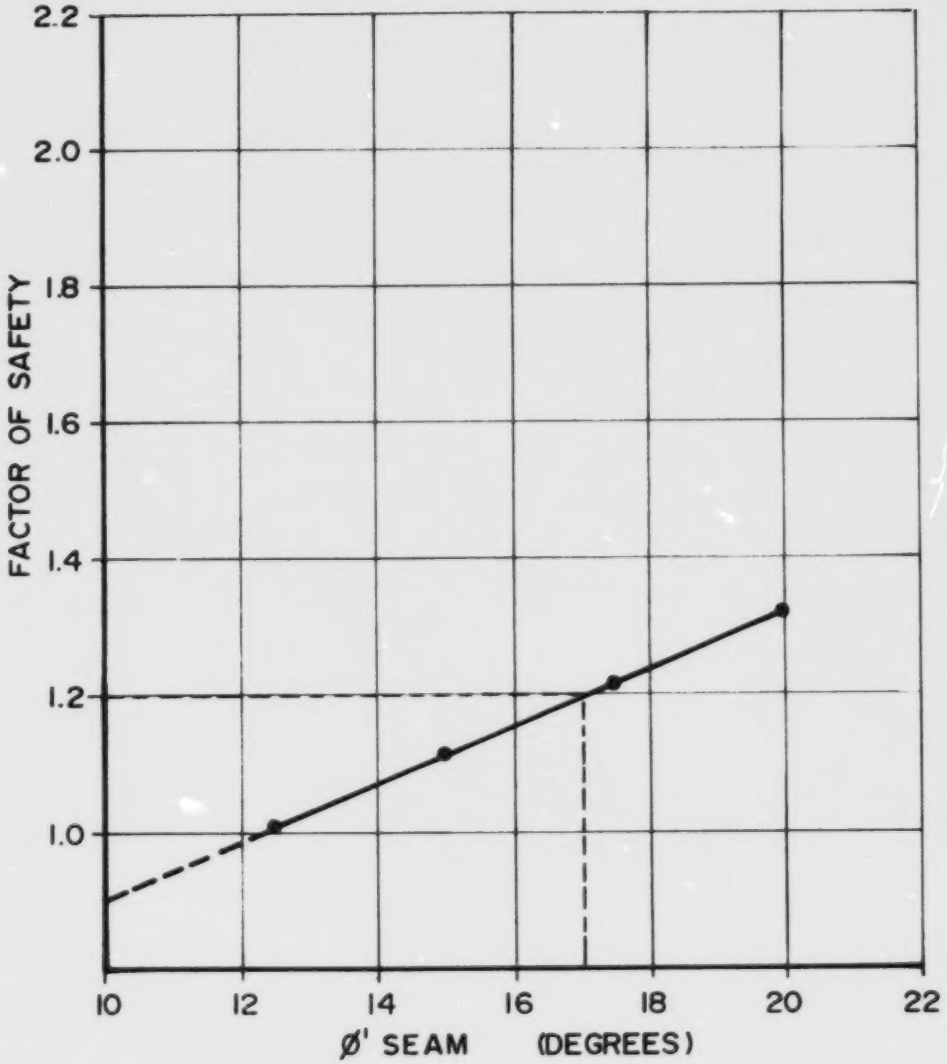
NOTE
SEAM V16 IS NOT CONTINUOUS IN DAM FOUNDATION

DAM - UPSTREAM SLOPE
FULL SUPPLY LEVEL
STEADY STATE SEEPAGE CONDITION
SLIDING THROUGH SEAM V16 IN UPPER CLAYSTONE OVERLYING BASAL SANDSTONE

OLDMAN RIVER DAM

FACTOR OF SAFETY VS Ø'
DAM - UPSTREAM SLOPE

PLATE C-7



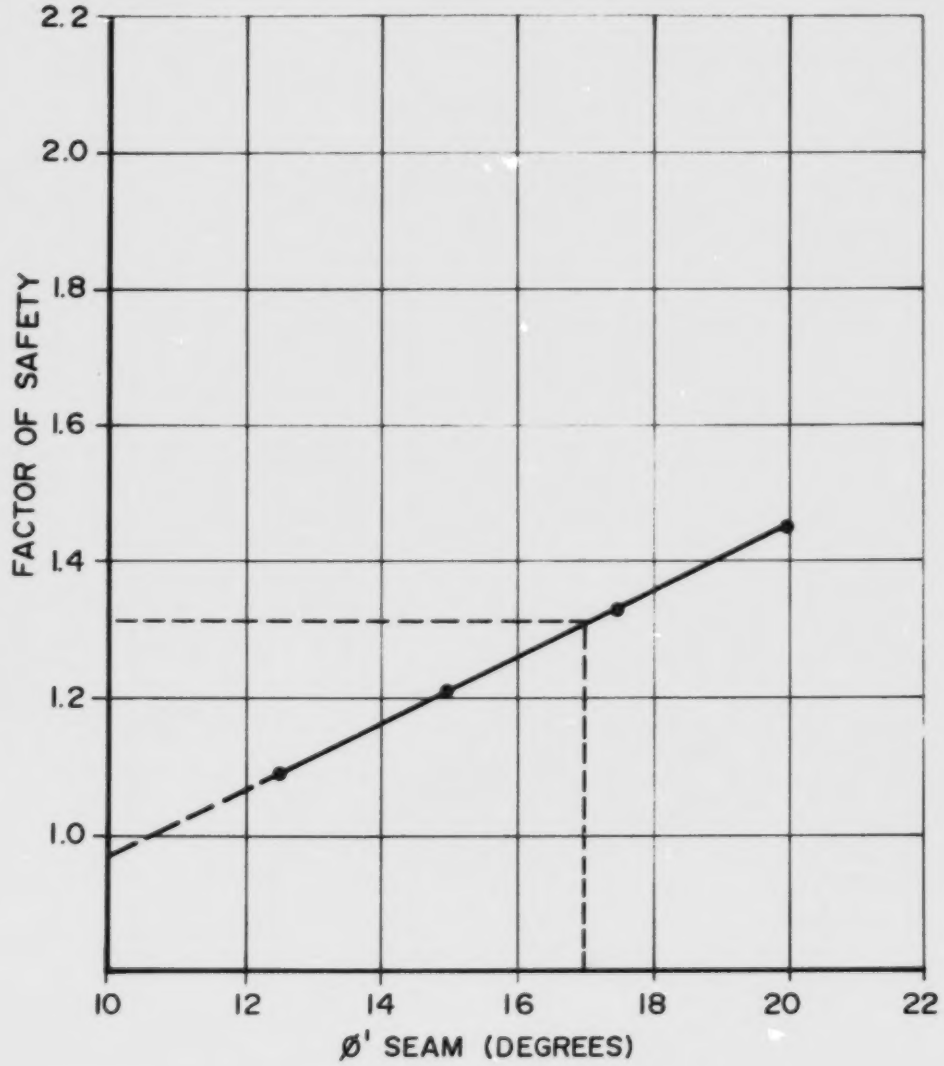
NOTE
SEAM V16 IS NOT CONTINUOUS IN DAM FOUNDATION

DAM-UPSTREAM SLOPE
FULL SUPPLY SLOPE
STEADY STATE SEEPAGE CONDITION
HORIZONTAL ACCELERATION=0.065g
SLIDING THROUGH SEAM V16 IN UPPER CLAYSTONE OVERLYING BASAL SANDSTONE

Alcoa of Australia
OLDMAN RIVER DAM

FACTOR OF SAFETY VS ø'
DAM - UPSTREAM SLOPE

PLATE C-8



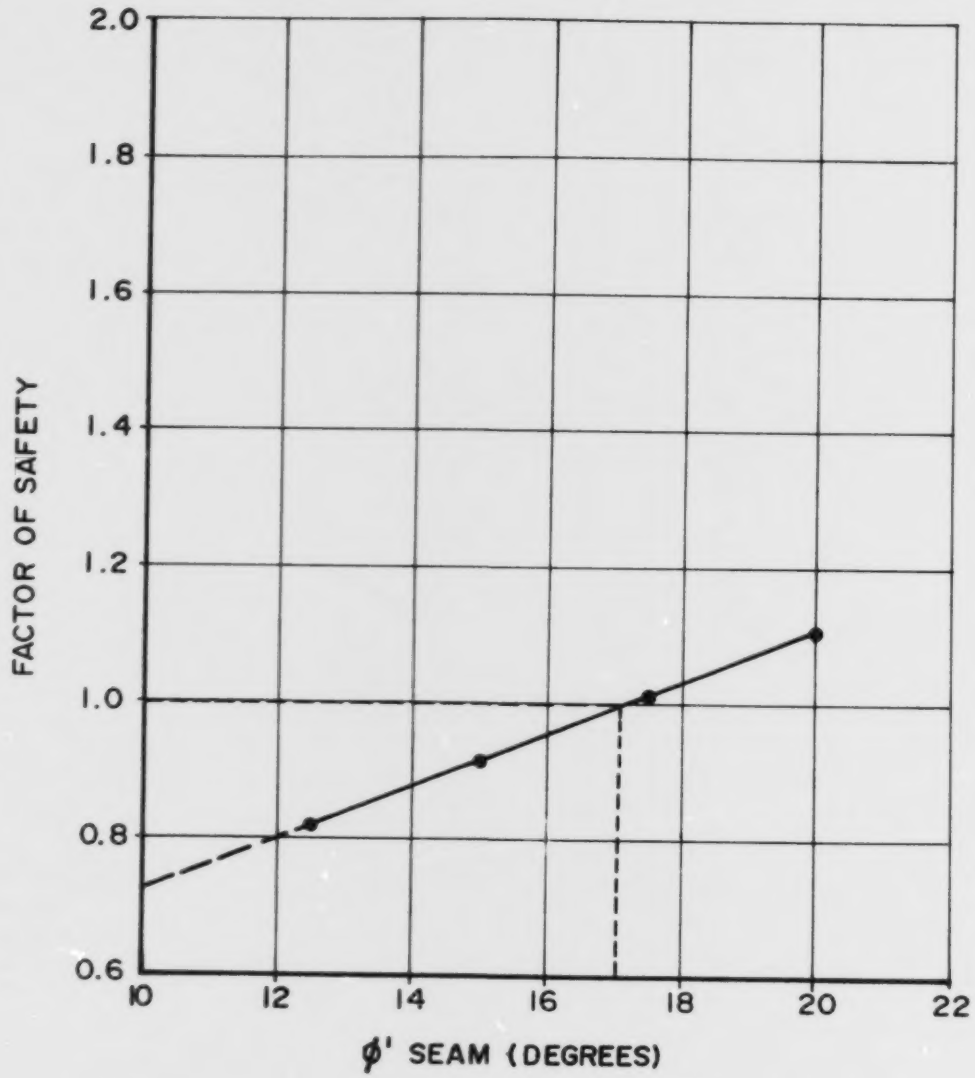
NOTE
SEAM V16 IS NOT CONTINUOUS IN DAM FOUNDATION

DAM - UPSTREAM SLOPE
RAPID DRAWDOWN
SLIDING THROUGH SEAM V16 IN UPPER CLAYSTONE OVERLYING BASAL SANDSTONE

OLDMAN RIVER DAM

FACTOR OF SAFETY VS Ø'
DAM - UPSTREAM SLOPE

PLATE C-9



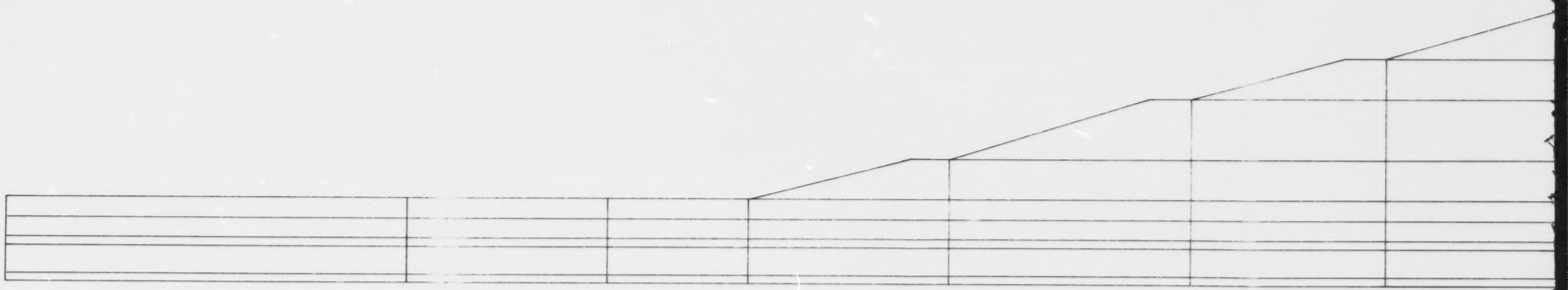
NOTE
SEAM V16 IS NOT CONTINUOUS IN DAM FOUNDATION

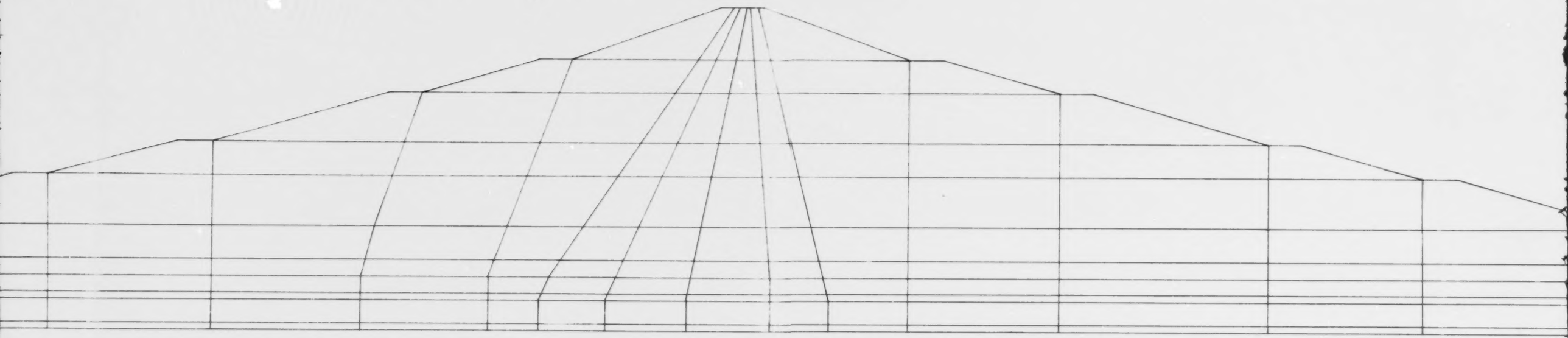
DAM – UPSTREAM SLOPE
RAPID DRAWDOWN
HORIZONTAL ACCELERATION = 0.065 g
SLIDING THROUGH SEAM V16 IN UPPER CLAYSTONE OVERLYING BASAL SANDSTONE

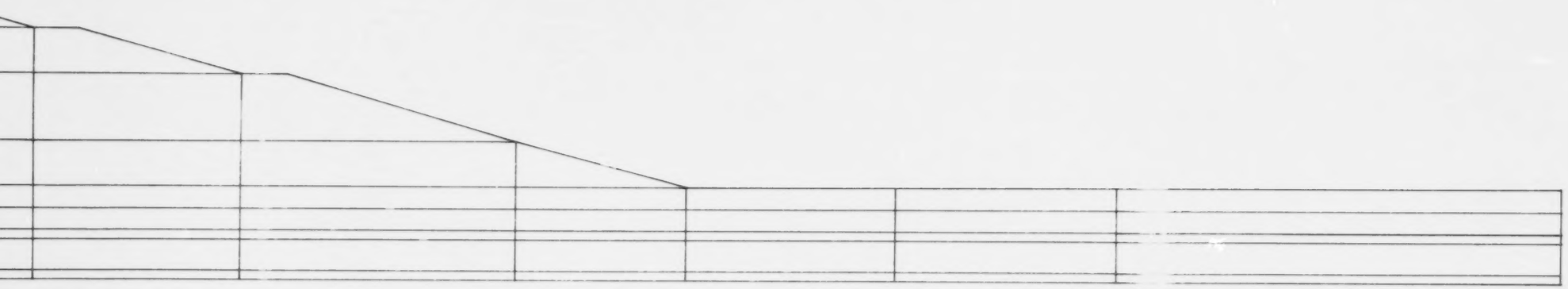
OLDMAN RIVER DAM

FACTOR OF SAFETY VS φ'
DAM - UPSTREAM SLOPE

PLATE C-10







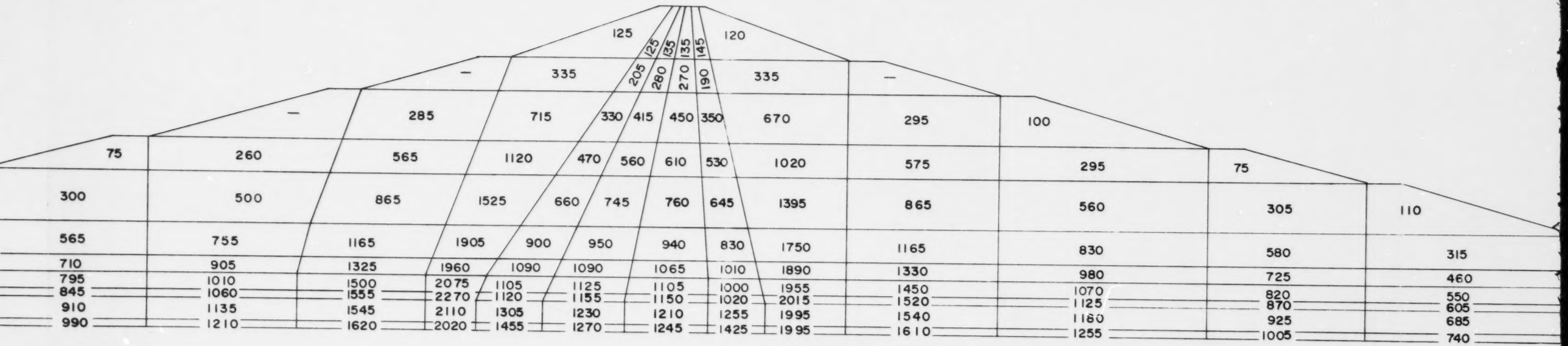
0 10 20 30 m
10

Alberta Environment
OLDMAN RIVER DAM

FINITE ELEMENT MESH

PLATE C-11

					75	260
				110	300	500
			75	310	565	755
35	35	40	175	455	710	905
105	115	145	275	545	795	1010
150	155	165	520	600	845	1060
240	285	375	470	670	910	1135
280	290	315	460	735	990	1210



VALUES OF σ_1' (kPa)

	75					
	305	110				
	580	315	65			
	725	460	160	40	35	35
	820	550	280	145	115	105
	870	605	315	165	155	150
	925	685	495	380	275	230
	1005	740	455	315	290	280

0 10 20 30 m

10

Alberta Power Limited
OLDMAN RIVER DAM

STATIC MAJOR PRINCIPAL EFFECTIVE STRESSES

PLATE C-12

					35	5
				60	125	22
			45	135	240	400
35	35	35	95	205	315	405
105	105	105	210	375	550	730
150	150	145	220	400	580	710
240	215	215	330	465	600	780
280	275	255	335	515	685	810

				20	65	110	100	35	15			
		—		0	140	160	135	65	0	75		
	95	—	130	235	255	220	135	105	35	20		
35	55	125	295	335	340	330	245	260	195	120	50	
125	220	390	480	440	440	435	405	460	375	235	140	65
240	400	695	800	555	575	570	575	680	555	355	245	140
315	405	570	730	710	695	680	715	755	575	425	320	205
550	730	950	815	775	765	750	730	825	940	725	550	375
580	710	920	1375	780	815	805	725	1325	975	750	580	400
600	780	650	400	100	150	210	290	305	625	660	550	460
685	810	1080	1375	1050	925	915	1045	1385	1100	860	690	510

VALUES OF σ_3' (kPa)

50					
140	65				
245	140	40			
320	205	85	35	35	35
550	375	190	105	105	105
580	400	205	150	150	150
550	460	320	215	215	215
690	510	320	255	270	275

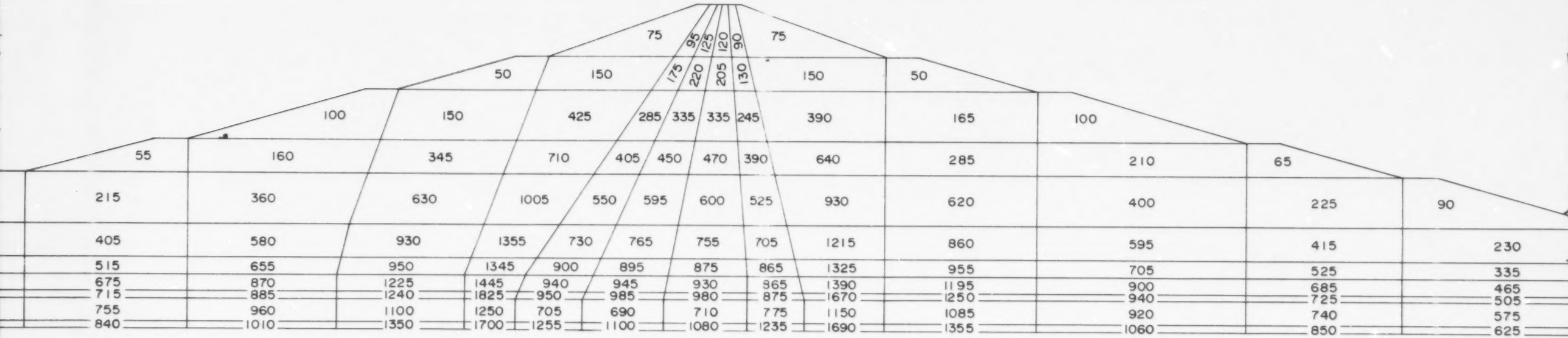
0 10 20 30 m
10

Alberta Transportation
OLDMAN RIVER DAM

STATIC MINOR PRINCIPAL EFFECTIVE STRESSES

PLATE C-13

				55		
			85	215	360	
			60	225	405	580
35	35	40	135	330	515	655
105	110	125	245	460	675	870
150	155	155	270	500	715	885
240	250	295	400	570	755	960
280	285	285	400	625	840	1010



VALUES OF $\sigma'_m = \frac{\sigma'_1 + \sigma'_3}{2}$ (kPa)

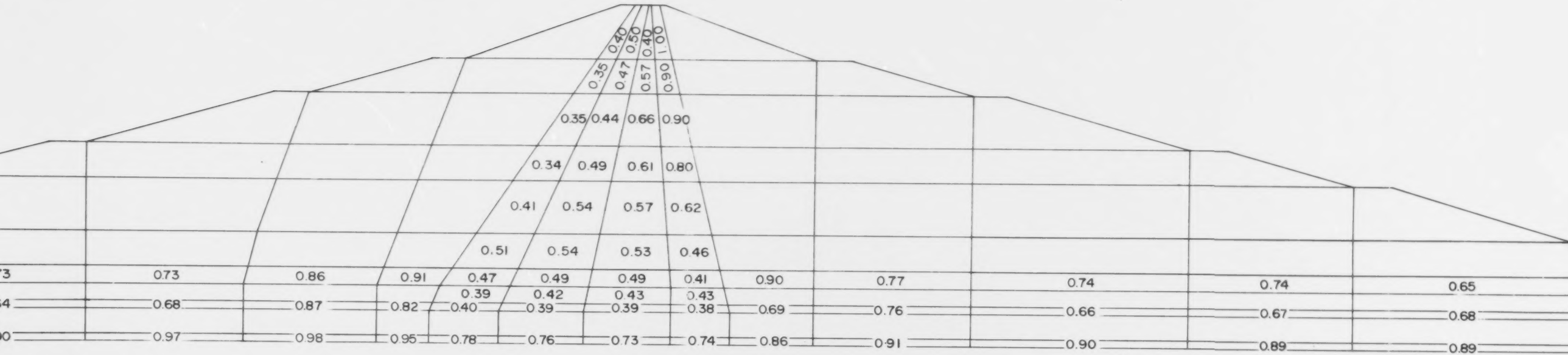
	65					
	225	90				
	415	230	55			
	525	335	125	40	35	35
	685	465	235	125	110	105
	725	505	260	160	155	150
	740	575	410	300	245	225
	850	625	390	285	280	280

Alberta Oil Sands Technology and Research Authority
OLDMAN RIVER DAM

STATIC MEAN
EFFECTIVE STRESSES

PLATE C-14

0.02	0.04	0.32	0.50	0.70	0.73	0.73
0.02	0.07	0.32	0.62	0.68	0.64	0.68
0.04	0.28	0.59	0.81	0.87	0.90	0.97



VALUES OF $(\sigma_1 - \sigma_3)/(\sigma_1 - \sigma_3)_f$

0.74	0.65	0.49	0.07	0.02	0.03
0.67	0.68	0.72	0.34	0.04	0.01
0.89	0.89	0.84	0.65	0.25	0.07

0 10 20 30 m

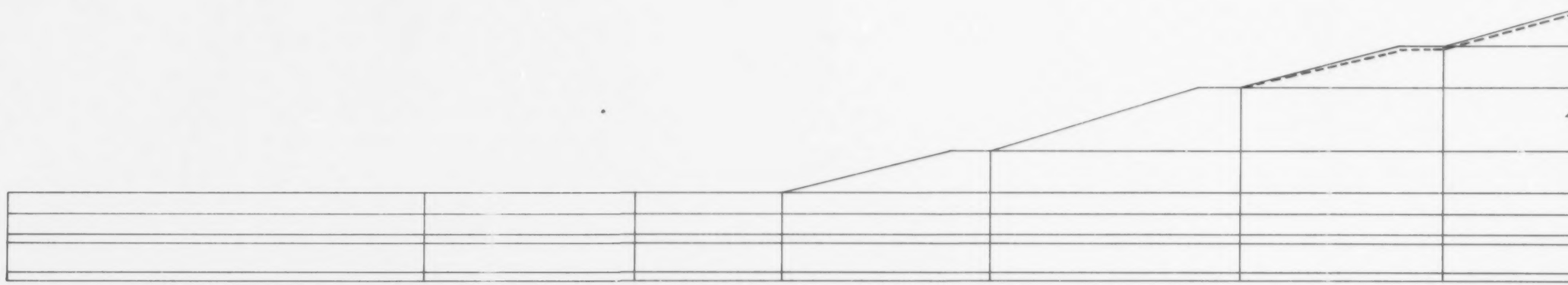
10

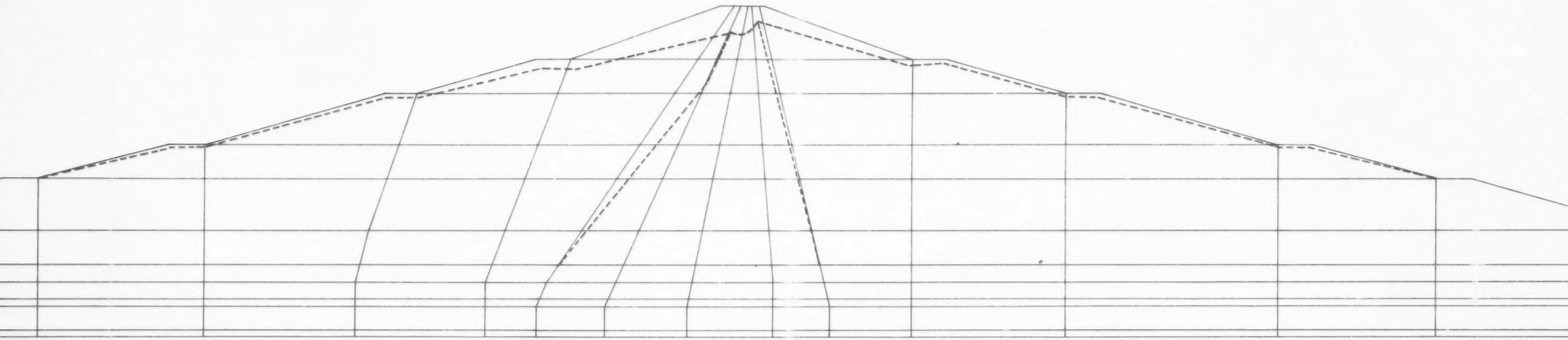
Acres International Limited

OLDMAN RIVER DAM

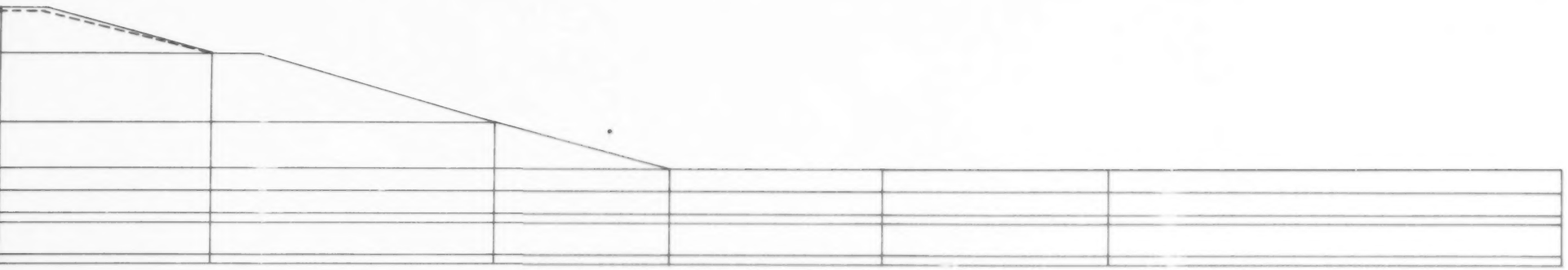
RATIO OF STATIC SHEAR STRESS TO STATIC SHEAR STRENGTH

PLATE C-15





DEFLECTED SURFACE PROFILE (5X)

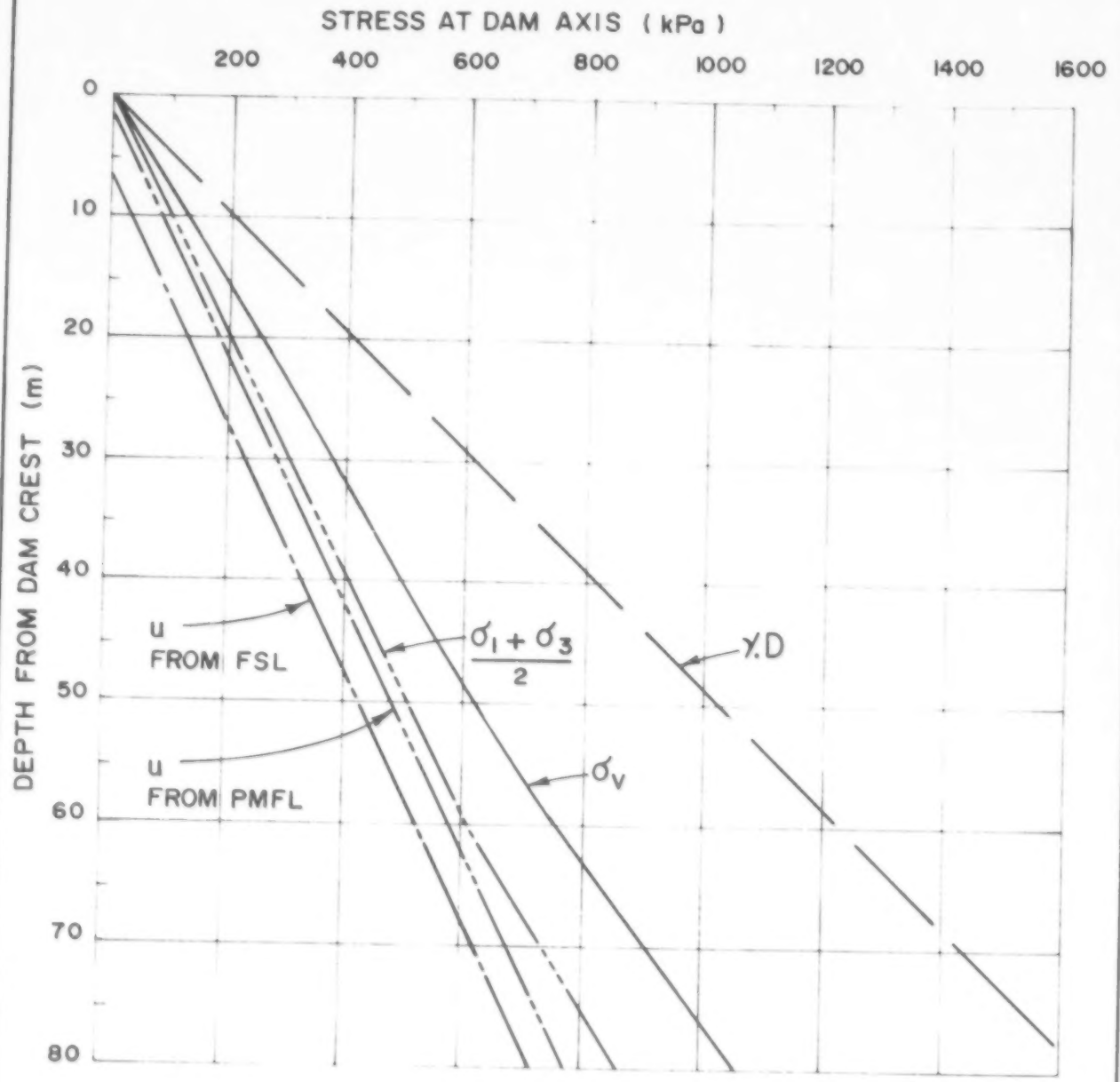


0 10 20 30 m

OLDMAN RIVER DAM

STATIC ELASTIC DEFLECTED SURFACE PROFILE

PLATE C-16



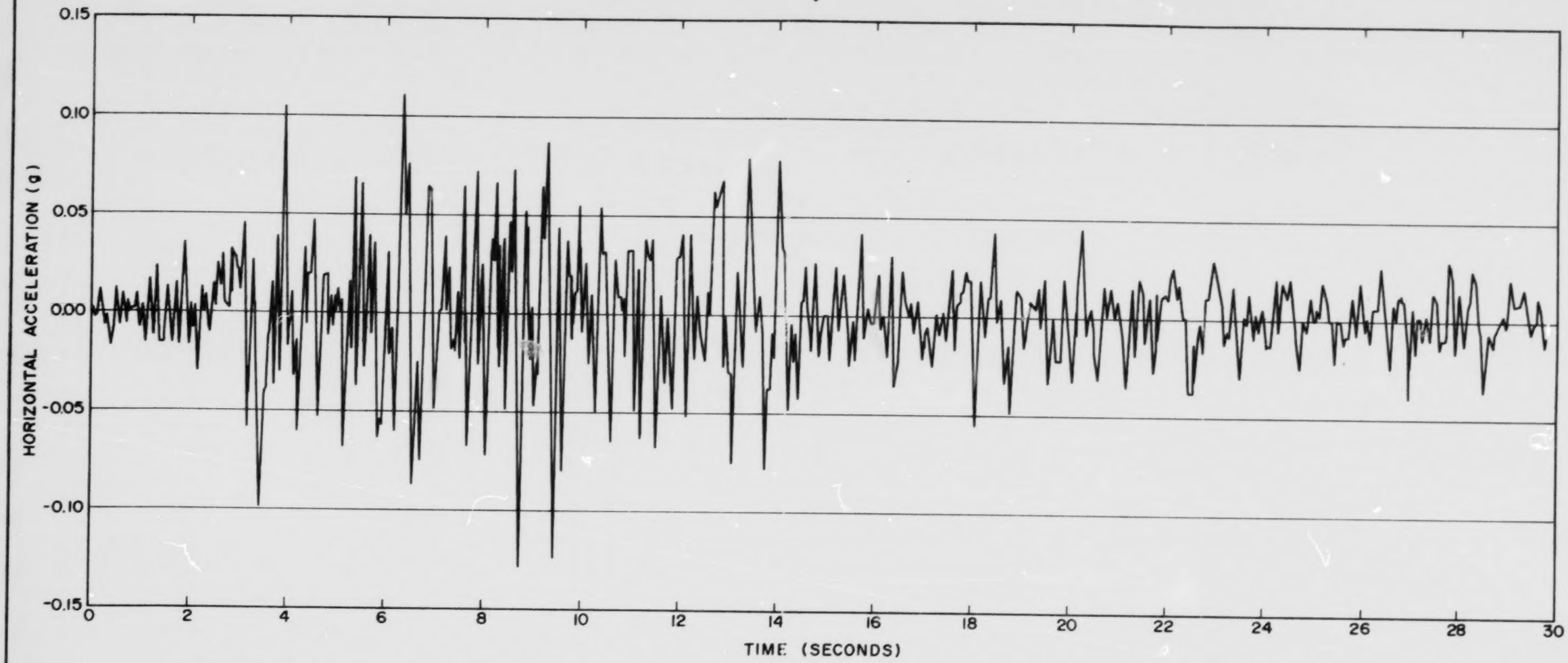
LEGEND

γ	– UNIT WEIGHT
D	– DEPTH FROM DAM CREST
σ_v	– TOTAL VERTICAL STRESS
σ_1	– TOTAL MAJOR PRINCIPAL STRESS
σ_3	– TOTAL MINOR PRINCIPAL STRESS
u	– PORE WATER PRESSURE
FSL	– FULL SUPPLY LEVEL
PMFL	– PROBABLE MAXIMUM FLOOD LEVEL

OLDMAN RIVER DAM

STATIC STRESSES AT DAM AXIS

PLATE C-17



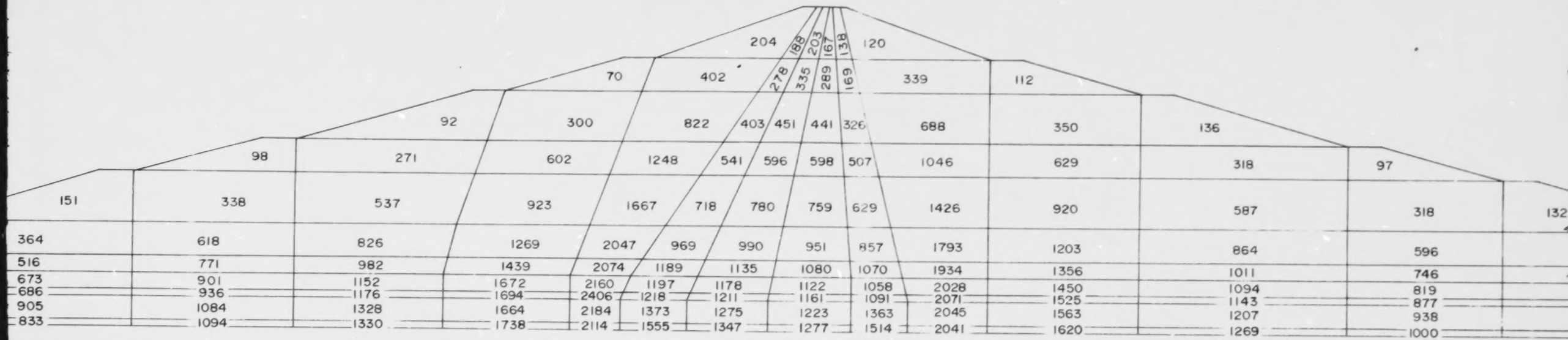
TAFT EARTHQUAKE ACCELERATION TIME HISTORY
SCALED TO A MAXIMUM ACCELERATION OF 0.13 g

Acres
OLDMAN RIVER DAM

DESIGN ACCELERATION
TIME HISTORY

PLATE C-18

						98	27
					151	338	537
				108	364	618	826
48	48	50	211	516	771	982	
162	190	233	417	673	901	1152	
198	196	206	388	686	936	1176	
333	479	624	770	905	1084	1328	
337	347	375	540	833	1094	1330	



VALUES OF COMBINED σ_1' (kPa)

97

318 132

596 326 84

746 475 163 49 48 48

819 557 365 229 187 162

877 603 296 191 194 198

938 816 780 642 474 334

1000 732 431 314 325 333

0 10 20 30m

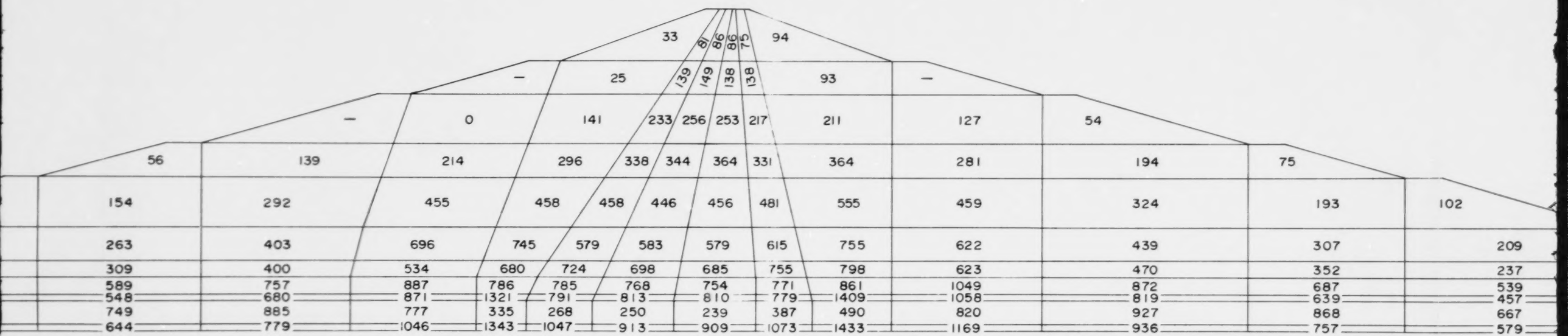
10

Acres International
OLDMAN RIVER DAM

COMBINED STATIC AND DYNAMIC MAJOR PRINCIPAL STRESSES

PLATE C-19

			56	139		
		74	154	292		
		56	161	263	403	
23	25	26	86	191	309	400
93	102	102	230	414	589	757
108	111	112	186	367	548	680
197	209	212	348	570	749	885
217	212	205	293	476	644	779



VALUES OF COMBINED σ_3' (kPa)

	75					
	193	102				
	307	209	68			
	352	237	107	27	25	23
	687	539	260	102	103	92
	639	457	256	130	114	108
	868	667	368	222	213	199
	757	579	381	264	234	221

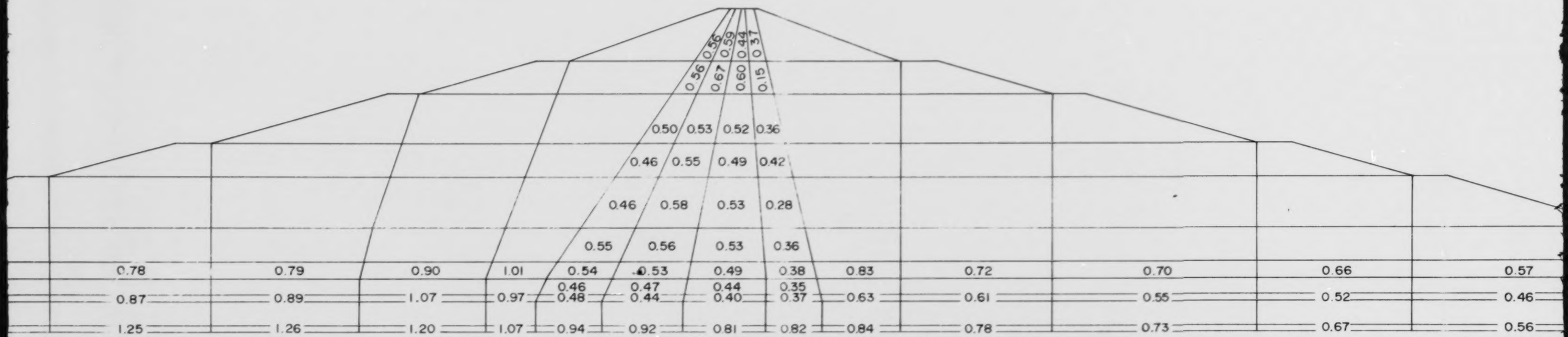
0 10 20 30 m
10

Alberta Power Limited
OLDMAN RIVER DAM

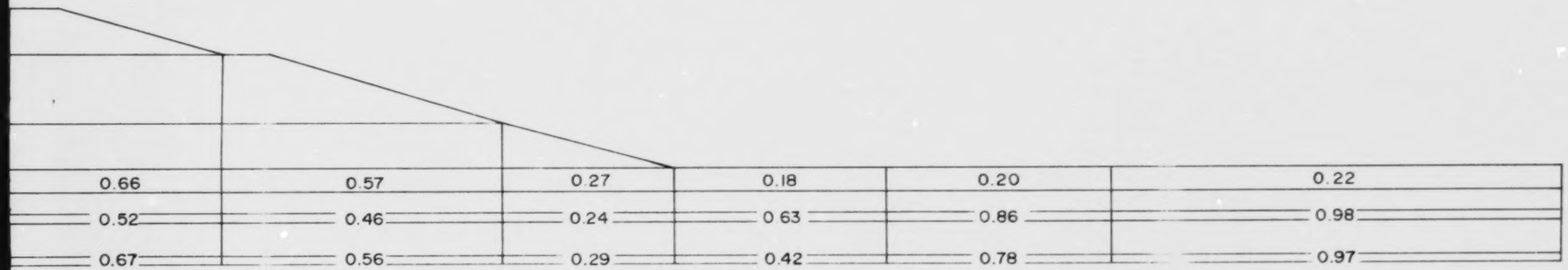
COMBINED STATIC AND DYNAMIC MINOR PRINCIPAL STRESSES

PLATE C-20

0.22	0.21	0.21	0.56	0.79	0.78	
0.98	0.92	0.99	1.17	1.01	0.87	
1.04	1.17	1.41	1.43	1.31	1.25	



VALUES OF $(\sigma_1 - \sigma_3)$ COMBINED / $(\sigma_1 - \sigma_3)_f$ STATIC

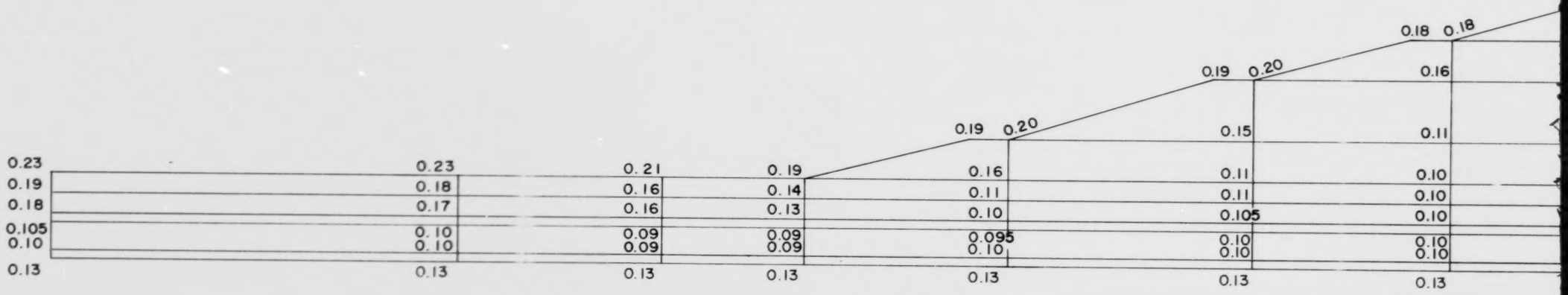


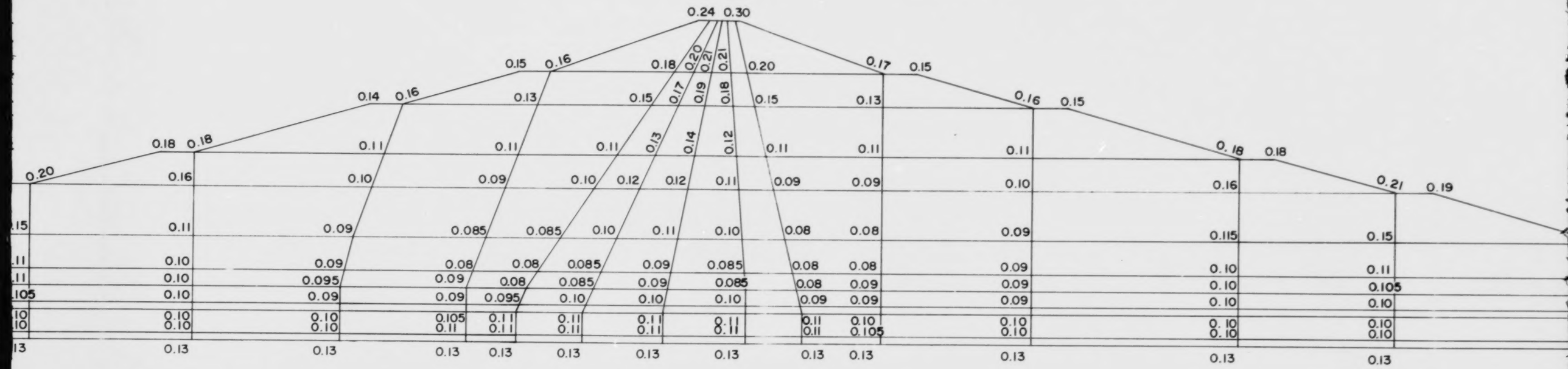
0 10 20 30 m

Alberta Umc
OLDMAN RIVER DAM

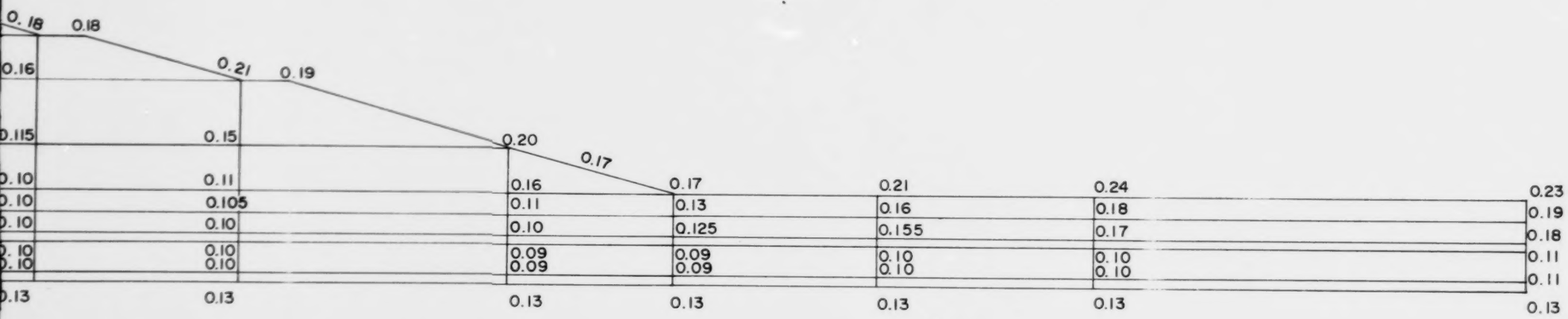
RATIO OF COMBINED STATIC AND DYNAMIC STRESSES TO STATIC SHEAR STRENGTH

PLATE C-21


0.23
0.19
0.18
0.105
0.10
0.13
0.23
0.18
0.17
0.10
0.10
0.13
0.21
0.16
0.16
0.10
0.09
0.09
0.13
0.19
0.14
0.13
0.09
0.09
0.13
0.19
0.20
0.16
0.11
0.10
0.095
0.10
0.13
0.19
0.20
0.15
0.11
0.11
0.105
0.10
0.10
0.13
0.18
0.18
0.16
0.11
0.10
0.10
0.10
0.10
0.13



VALUES OF a_{max} (g)



0 10 20 30 m

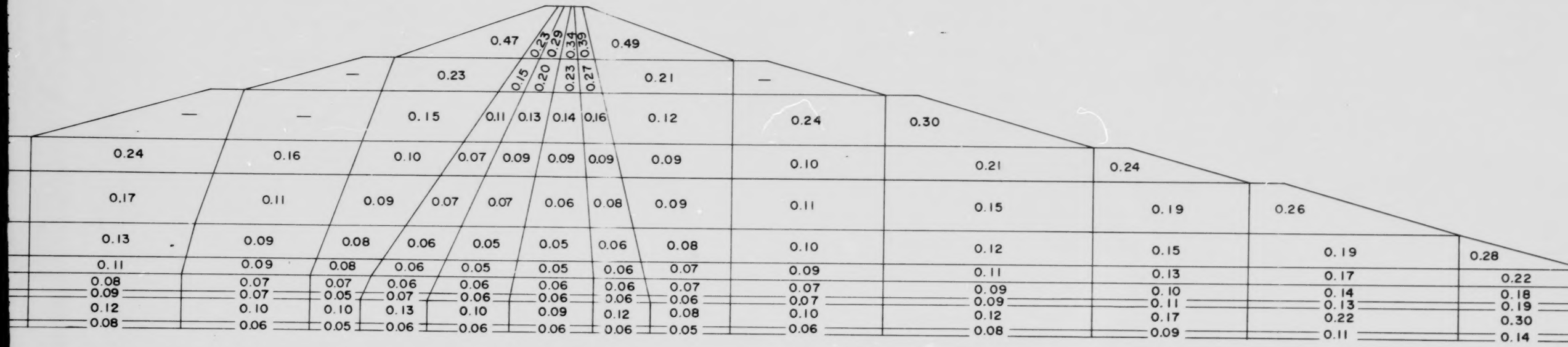
10

Alberta Environment
OLDMAN RIVER DAM

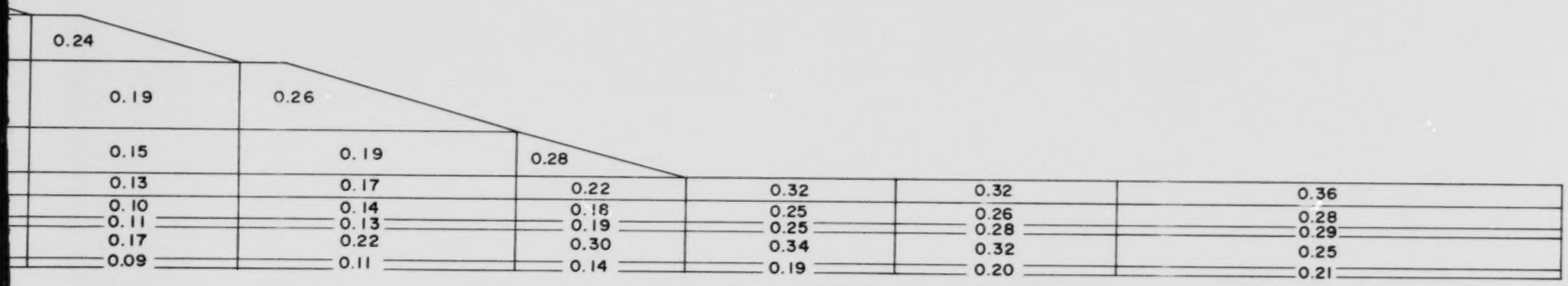
MAXIMUM HORIZONTAL ACCELERATIONS AT NODES

PLATE C-22

							—
						0.28	0.24
					0.26	0.20	0.17
				0.26	0.18	0.15	0.13
0.36		0.32	0.34	0.20	0.17	0.13	0.11
0.28		0.26	0.25	0.18	0.14	0.10	0.08
0.29		0.28	0.25	0.18	0.13	0.11	0.09
0.25		0.32	0.34	0.29	0.22	0.16	0.12
0.21		0.20	0.19	0.14	0.11	0.09	0.08



VALUES OF $(\sigma_1 - \sigma_3)_d \; / \; 2\sigma m_{(s\ \&\ d)}$



0 10 20 30m
10

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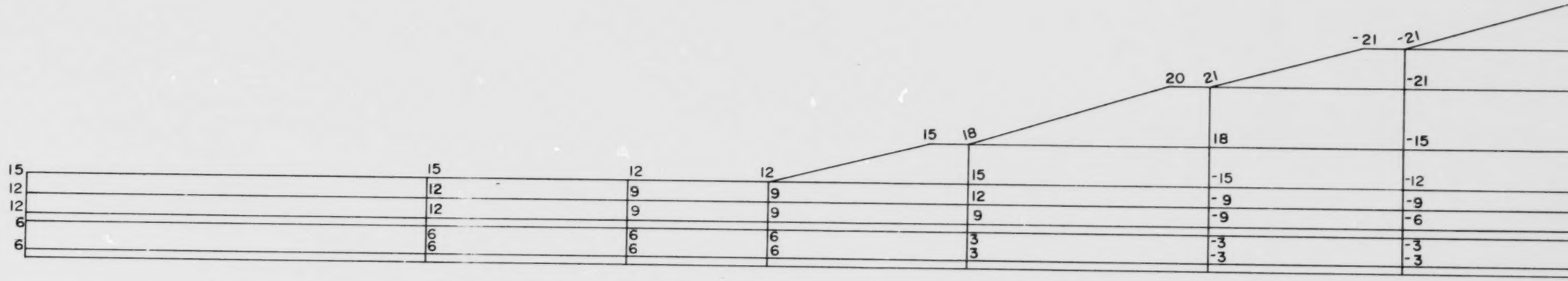
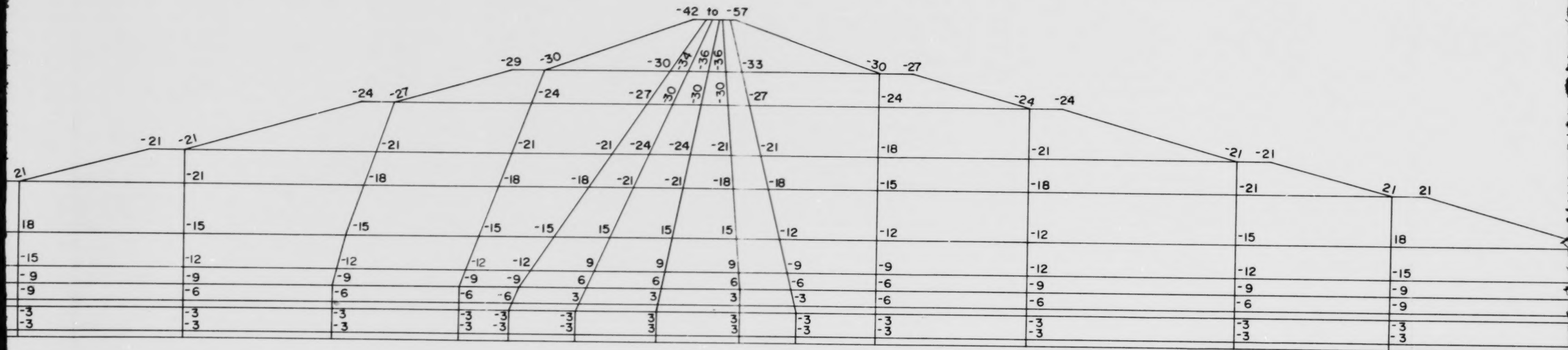
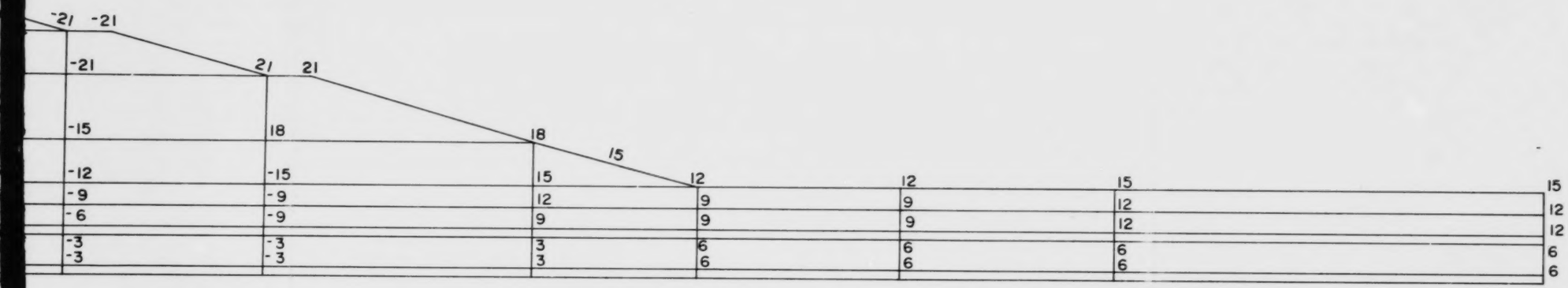
RATIO OF MAXIMUM DYNAMIC SHEAR STRESS TO COMBINED STATIC & DYNAMIC MEAN STRESS

PLATE C-23


15 15 12 12 15 18 20 21 -21 -21
15 15 12 12 15 18 -21 -21
12 12 9 9 12 15 -15 -15
12 12 9 9 9 9 -9 -9
6 6 6 6 3 3 -3 -3
6 6 6 6 3 3 -3 -3



MAXIMUM DISPLACEMENTS (mm)



0 10 20 30 m
10

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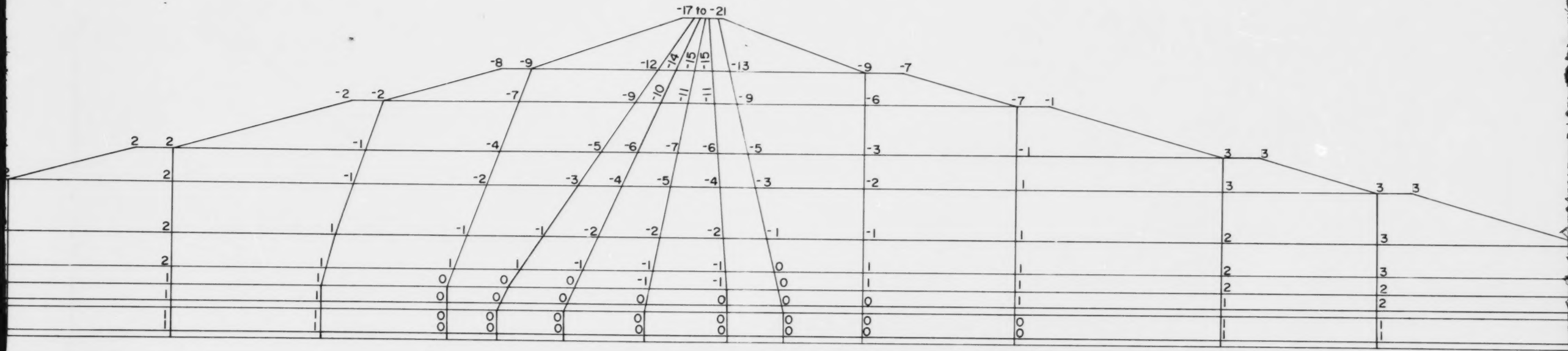
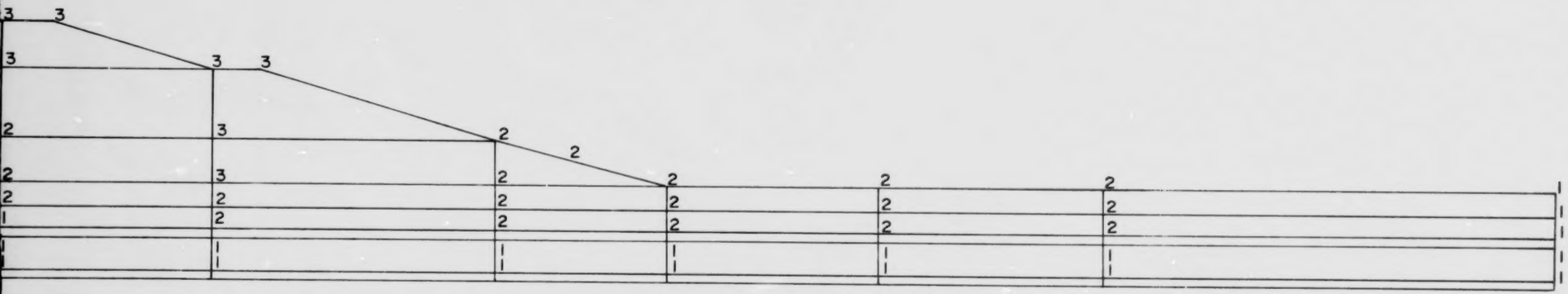
OLDMAN RIVER DAM

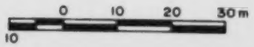
MAXIMUM DYNAMIC
NODAL DISPLACEMENTS

PLATE C-24



FINAL DISPLACEMENTS (mm)





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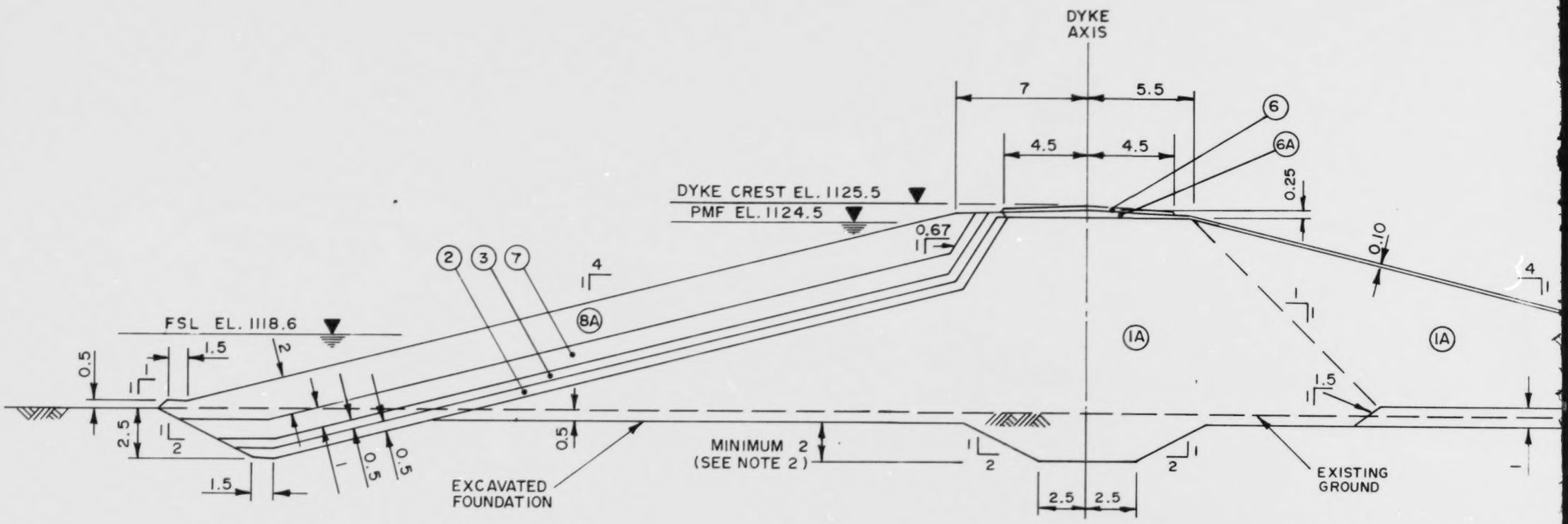
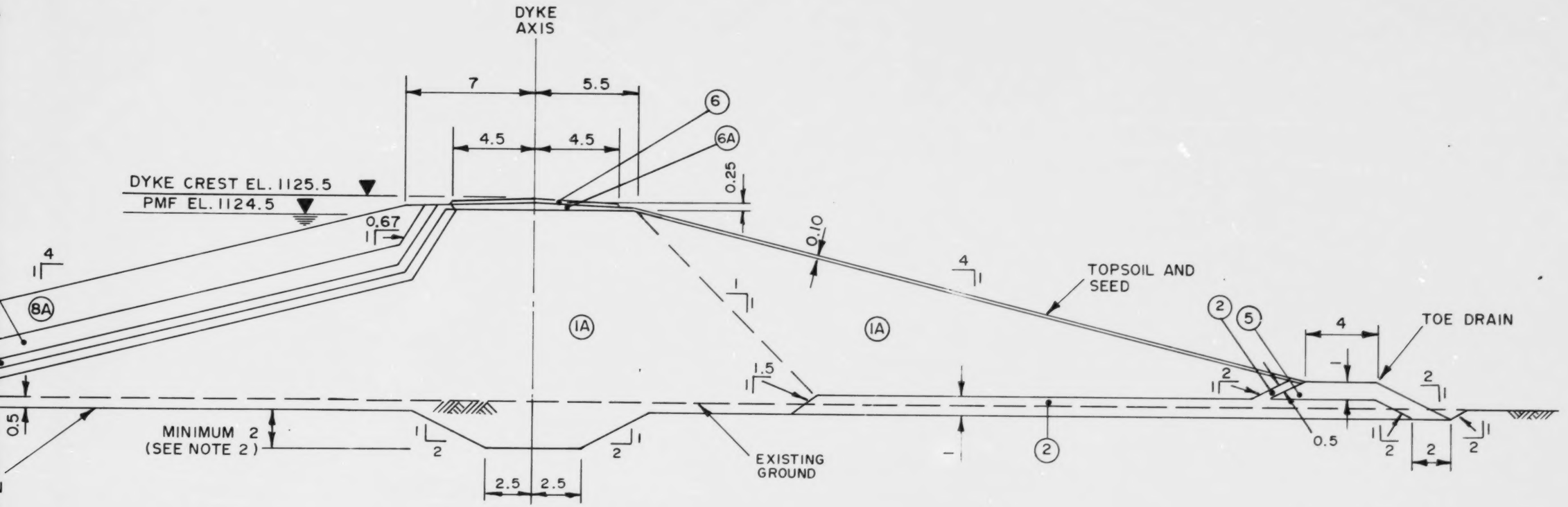
FINAL DYNAMIC
NODAL DISPLACEMENTS

PLATE C-25


DYKE
AXIS
7
5.5
4.5
4.5
6
6A
DYKE CREST EL. 1125.5
PMF EL. 1124.5
0.25
0.10
0.67
1
4
1
2
3
7
8A
FSL EL. 1118.6
1.5
2
0.5
1
1
2.5
1
2
1.5
0.5
0.5
0.5
EXCAVATED
FOUNDATION
MINIMUM 2
(SEE NOTE 2)
1
2
1
2
1A
1A
1
1
1.5
1
EXISTING
GROUND
2.5
2.5



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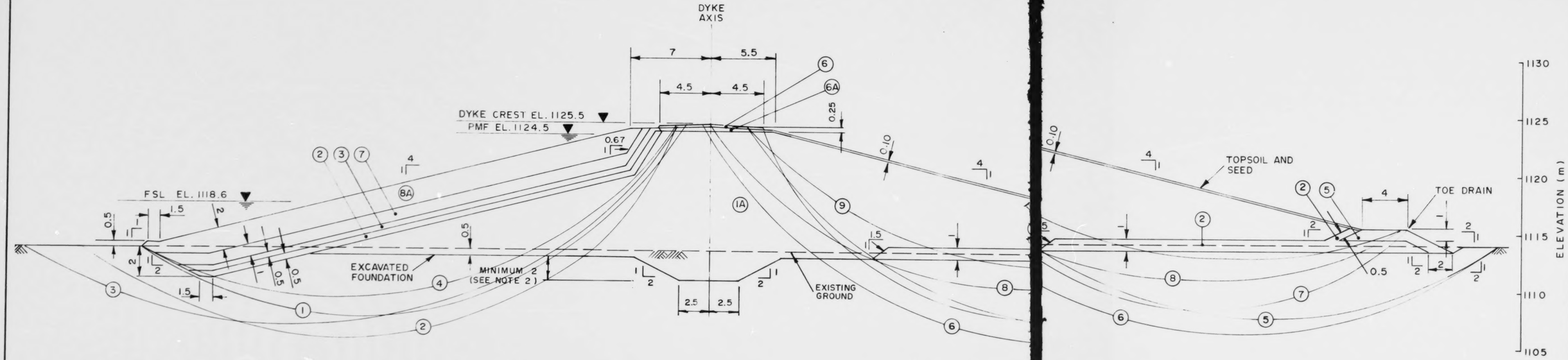
DYKE SECTION

PLATE C-26



NOTE:
FOR FACTORS OF SAFETY REFER TO SECTION C-7, TABLE C-4.

1:200 HORZ

Alcan
OLDMAN RIVER DAM

DYKE STABILITY ANALYSIS
POTENTIAL SLIP SURFACES

PLATE C-27

OLDMAN RIVER DAM

DESIGN REPORT

APPENDIX D

STABILITY OF EXCAVATED AND EXISTING SLOPES

SEPTEMBER, 1988

APPENDIX D

STABILITY OF EXCAVATED AND EXISTING SLOPES

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D.1 STABILITY OF EXCAVATED AND EXISTING SLOPES

D.1.1 General

Stability analyses have been carried out for the design of the excavated slopes and to check the stability of the existing slopes adjacent to the major structures. The excavated slopes include those along the spillway approach channel, chute and flip bucket and the overburden slopes upstream and downstream of the dam. The existing rock slopes upstream and downstream of the dam have also been analyzed. The location of the slopes analysed is shown on Plate D-1.

The stability analyses have considered deep seated failure surfaces only. Slaking of the mudrocks in the slopes will cause local shallow sloughing of all bedrock slopes during the life of the project.

D.1.2 Geology

The excavated and existing slopes are in the Upper Mudrock sequence which consists of interbedded sandstone, siltstone and claystone layers. The bedrock is overlain by Quarternary surficial deposits of glacial till and glaciolacustrine silt and clay.

A simplified stratigraphy for each slope cross section analyzed was developed from nearby drillhole and surface exposure data. The stratigraphy is generally composed of glaciolacustrine silt and clay, clay till, weathered mudrock, slightly weathered mudrock, intact mudrock and sandstone. Mudrock includes both siltstone and claystone. In locations where the mudrock is composed of several thin zones of interbedded weathered and intact rocks, it has been categorized as being slightly weathered. An

appropriate effective cohesion intercept was assigned to the shear strength of these zones to reflect the relative amount of weathered and unweathered rock and the proportions of sandstone, siltstone and claystone in the zones.

Several subhorizontal major seams exist in the mudrock as discussed in Appendix B. These seams were considered in the analyses by modelling a flat segment of the sliding planes to pass through them.

A small normal fault plane was found in the flipbucket area, immediately above one of the two test anchors. It trends approximately north-south, dips 50^{o} west. Discontinuous clay gouge up to 6 mm thick was found on the fault plane. Approximately 350 mm of dip-slip offset was found. This is a feature not characteristically found in any of the other excavations at site. Physically it resembles an inclined joint plane of through going continuity, which could lead to local instability of unfavorable orientated excavation cuts.

If, during construction other similar features are encountered, their site specific aspects will be obtained from geological mapping. Appropriate design modifications will then be implemented which may include:

- Local modification of excavation limits to avoid the feature.
- Removal of unstable materials.
- Additional rock support and stabilization measures.

D.1.3 Material Properties for Stability Analyses

The shear strength and unit weight parameters used in the analyses are listed in Tables D-1 to D-3 inclusive. These parameters have been derived from a geotechnical and the geological assessment of the strength of the foundation bedrock and the seams found in the rock and the results of the laboratory testing program described in Appendix B.

Overburden

The effective angle of shearing resistance and the unit weight assigned for the glaciolacustrine silt and clay and clay till have been based on the index and shear strength test results as discussed in the Preliminary Engineering Report, (UMA/Acres (1986)). A small effective cohesion was used in the slope stability analyses to establish circular sliding planes entirely in the overburden. The shear strength of the overburden is given in Table D-1.

Bedrock

Three sets of shear strength parameters are required for the bedrock stability analysis. These comprise the shear strength of the rock across the bedding planes, the shear strength in the seams along the bedding planes and the shear strength along the bedding planes in the intact rock between the seams.

The cross-bedding effective angle of shearing resistance selected for each of the bedrock types was based on the laboratory test results as discussed in the Preliminary Engineering Report, (UMA/Acres (1986)), and Appendix B. An effective cohesion

component of the cross-bedding shear strength of some of the mudrocks was used in stability analyses. The effective cohesion was selected on the basis of weathering and rock type. For the sandstone, and for weathered, brecciated, slickensided and highly fractured mudrock zero effective cohesion was assumed. For the unweathered intact mudrock an effective cohesion of 200 kPa was used. For the slightly weathered mudrocks, the effective cohesion assigned varied between 0 and 200 kPa depending on the degree of weathering and on the relative amount of siltstone and claystone recorded in the adjacent drillholes and surface exposures. The cross-bedding shear strengths are given in Table D-1.

The shear strength in the design seams along the bedding planes and the shear strength along the bedding planes between the design seams are discussed in Appendix B. The effective cohesion along all seams is zero. Their values are given in Tables D-2 and D-3.

TABLE D-1

SHEAR STRENGTH OF OVERBURDEN

AND

CROSS-BEDDING SHEAR STRENGTH IN BEDROCK

MATERIAL	EFFECTIVE COHESION c' (kPa)	EFFECTIVE ANGLE OF SHEARING RESISTANCE ø'	UNIT WEIGHT (kN/m^3) γ
OVERBURDEN			
- Glaciolacustrine Silt & Clay	5	22°	21.6
- Clay Till	15	25°	21.6
BEDROCK			
- Sandstone	0	60°	23.6
- Intact Mudrock	200	35°	23.6
- Slightly weathered mudrock	*	35°	23.6
- Weathered, Brecciated, Slickensided and Badly Fractured Mudrock	0	35°	23.6

* Effective cohesion varies between 0 and 200 kPa depending on the degree of weathering and the relative amount of siltstone and claystone.

TABLE D-2

SHEAR STRENGTH ALONG DESIGN SEAMS

DESIGN SEAM NO.	REFERENCE ELEVATION* (m)	DESIGN EFFECTIVE ANGLE OF SHEARING RESISTANCE	UNIT WEIGHT (KN/m^3)
V12	1030.7	12.5^o	23.6
V16	1046.8	17.0^o	23.6
V18	1049.4	16.0^o	23.6
R1	1068.2	11.5^o	23.6
R6	1084.5	13.5^o	23.6
R7	1086.5	13.0^o	23.6
R8	1087.5	12.5^o	23.6
R9	1088.5	11.5^o	23.6
R13	1096.5	15.5^o	23.6
R14	1099.8	16^o	23.6
R16	1105	13^o	23.6
R17	1107.7	16.5^o	23.6
R18	1115	12.5^o	23.6

* For seams V12, V16 and V18 the reference elevation is dam axis. For the remaining seams, the reference elevation is spillway centreline.

TABLE D-3

SHEAR STRENGTH ALONG BEDDING PLANES BETWEEN DESIGN SEAMS

ELEVATIONS (m)	DESIGN EFFECTIVE ANGLE OF SHEARING RESISTANCE	UNIT WEIGHT (KN/m^3)
Below El. 1040	17.5	23.6
El. 1040 to El. 1050	17.5	23.6
El. 1050 to El. 1085	17.5	23.6
El. 1085 to El. 1090	14	23.6
El. 1090 to El. 1100	17.5	23.6
Above El. 1100	17.5	23.6

D.1.4 Pore Pressure Parameters

The phreatic surface for the steady state seepage condition was determined from the finite element analyses of seepage through the abutments described in Appendix E. The permeability of the overburden and bedrock is variable but could be low in significantly large zones particularly in the weathered layer at the surface. It has therefore been assumed that water will drain from the slopes slowly. Drainage will be primarily through fissures in the clay till and weathered and fractured zones in the bedrock. With this uncertainty in mind the phreatic surface for the rapid drawdown condition was assumed to be close to that for the steady state condition at full supply level within the slope and to generally follow the surface of the slope some distance within the slope to allow for the pore pressure dissipation that will take place. The actual phreatic surfaces assumed are shown on Plates D-2 to D-21 inclusive.

D.1.5 Method of Analysis

The slope failure mechanisms considered most appropriate for the topography and rock and soil types found at the project site are:

- Sliding on a near circular surface where the sliding surface passes entirely in overburden.

- Sliding on a composite surface through the overburden and bedrock, with the horizontal segment of the sliding surface passing through the bedrock and overburden contact.

- Sliding on a composite surface through the overburden and bedrock or through the bedrock only, where the overburden is not present, with the horizontal segment of the sliding surface passing through seam parallel to the bedding planes.

Sensitivity analyses were done for sections using sliding surface passing through seams above, at and below the toe of the slope. These analyses showed that the critical sliding surface always passes through the toe of the slope. Thus, in the stability analyses, it has been assumed that a seam exists at the toe of the slope. However, the stability was checked for higher seams when they have a lower effective angle of shearing resistance.

Stability analyses were carried out using the slope stability program "PCSTABL4". The program calculates the factor of safety against slope failure by a two dimensional limit equilibrium method. The calculation of the factor of safety against slope instability is performed by the "Modified Janbu" method of slices which allows the analysis of non-circular composite sliding surfaces as well as those of circular shape. This method is particularly suited to analysis of composite sliding planes which are constrained to follow particular paths such as along seams in bedrock (Fredlund and Krahn (1977), Fell et al (1987), Janbu (1973), Janbu & Grande (1976), Bishop (1955), Nonveiller (1965), Morgenstern & Price (1965), Eigenbrod & Morgenstern (1972)).

The slopes have been designed for the required minimum safety factors given in Table D-4.

D.1.6 Three-Dimensional Factor of Safety

The assumption of two-dimensional, plane strain, geometry is common to all limit equilibrium slope stability calculations, yet no landslide ever occurs in plane strain. However, end effects and the three-dimensional shape of the sliding surface are usually neglected for convenience compress the slope stability problem into the limited framework of a two-dimensional model. Three-dimensional effects may not be too important depending on the geometry of potential sliding surfaces, the type of soil or

rock and the presence of adjacent slope constraints. Because of the complex geometry and geology of the excavated and existing slopes at the project site, it is necessary to account for such effects otherwise the design would be unnecessarily conservative and unrealistically costly.

TABLE D-4

REQUIRED MINIMUM FACTORS OF SAFETY

STABILITY CASE	REQUIRED MINIMUM FACTOR OF SAFETY
1. Reservoir at full supply level and steady state seepage conditions.	1.5
2. Reservoir at full supply level and steady state seepage with horizontal pseudo-static seismic loading 0.065 g.	1.0
3. Rapid reservoir drawdown from full supply level to minimum supply level.	1.3
4. Rapid reservoir drawdown from full supply level to minimum supply level with horizontal pseudo-static seismic loading 0.065 g (for upstream slopes only).	1.0

There is no general method available for three-dimensional analysis of slopes involving sliding surfaces of arbitary shape. Only a few attempts at three dimensional slope stability analysis have been published. Among these are the work done by Sherard et al (1963), Baligh and Azzouz (1975), Hovland (1977), Chen and Chameau (1982) and Hungr (1987).

Studies by Baligh and Azzouz (1975) show that for a cohesive soil, consideration of end effects can lead to a 4 percent to 40 percent increase in the factor of safety depending upon the shape of the sliding surface which in turn is a function of the amount of cohesion in the slope material.

Results published by Hovland (1977), Chen and Chameau (1982) and Hungr (1987) indicate that the three-dimensional factor of safety can be from 0 to 40 percent higher than the two-dimensional factor of safety depending on the material properties, the shape of the three-dimensional sliding surface, and the pore pressures.

As all of the published data show a wide variation in the ratio of the three-dimensional factor of safety to the two-dimensional factor of safety, a ratio has been selected for the design of individual slopes that is considered appropriate for the conditions that exist in that slope. In no case is the selected ratio greater than ten percent.

D.1.7 Results of Analysis

D.1.7.1 River Slopes

Plates D-2 through D-13 illustrate the sections analyzed and the factors of safety obtained.

Plate D-2 shows the section through the overburden excavation on the left bank upstream of the dam axis. Circular sliding surfaces through the overburden and composite sliding surfaces through the bedrock and overburden contact have been analyzed. The composite sliding surfaces passing through the bedrock and overburden contact are the most critical. The slope will be excavated to 4

horizontal to 1 vertical to achieve the required minimum factor of safety of 1.3 for the rapid reservoir drawdown case. All other stability cases have factors of safety higher than or equal to the required minimum factors of safety.

Plate D-4 illustrates the section through the existing rock slope on the left bank upstream of the dam axis. The section is taken at a location where the rock slope has a maximum height and has the steepest slope. The channel sandstone will act as a drain during rapid reservoir drawdown. Two sets of sliding surfaces have been considered. The critical sliding surface will be through the toe of the slope along seam V18 at elevation 1055 m. The safety factor for a sliding surface passing along this seam is lower than for sliding surfaces passing through seam R9 at elevation 1082 m. Except for the case No. 2, rapid reservoir drawdown from full supply level to minimum supply level, all factors of safety based on the two dimensional considerations are equal to, or higher than the required minimum factors of safety. Adding ten percent to the calculated factor of safety for case No. 2 to allow for three dimensional effects will bring it up to the minimum required value. The existing rock slopes will therefore remain stable.

Plate D-6 illustrates the section through the overburden excavation on the right bank upstream of the dam axis. Circular sliding surfaces entirely through the overburden and composite sliding surfaces through the rock and overburden contact have been analyzed. The composite sliding surface through the bedrock and overburden contact is the most critical. The slope will be excavated 3.5 horizontal to 1 vertical to achieve the required minimum factor of safety for steady state seepage with the reservoir at full supply level. This slope will have the minimum factors of safety for rapid reservoir drawdown and rapid reservoir drawdown with pseudo-static seismic loading.

Plates D-8 and D-12 illustrate the sections through the right bank and left bank overburden excavations at downstream of the dam axis respectively. Circular sliding surfaces entirely through the overburden and composite sliding surfaces through the overburden and rock contact have been analyzed. The composite sliding surfaces through the bedrock and overburden contact is the most critical. The slopes will be excavated to 3 horizontal to 1 vertical to achieve the required minimum factors of safety for steady state seepage.

Plate D-10 illustrates the section through the existing rock slope on the right bank downstream of the dam centerline. The section has been taken at the location where the height of the slope is maximum. Two sets of sliding surfaces have been analyzed. One set passes through the toe of the slope. The second set of sliding surfaces passes through seam R9 at elevation 1086 m. For all cases, the factors of safety are higher than or equal to the required minimum factor of safety, therefore the existing slopes will remain stable.

The effect of a possible lower effective angle of shearing resistance existing along the seams, on the stability of the existing river slopes, has been examined with sensitivity analyses of the relationship between the factor of safety and the effective angle of shearing resistance of the seam. Plates D-3, D-5, D-7, D-9, D-11 and D-13 inclusive show the results of these analyses.

D.1.7.2 Approach Channel and Spillway Slopes

Approach Channel

The results for the approach channel slopes are shown on Plates D-14 and D-15. Plate D-14 illustrates the section analyzed and the factors of safety obtained. The section analyzed is the

deepest section in the approach channel. Sliding surfaces through seams R14, R15, R16 and R17 at elevations 1099, 1102, 1105 and 1107.7 m were considered. The critical sliding surface passes through seam R14 at the toe of the excavation.

To obtain the required minimum factors of safety the approach channel rock slopes will be excavated to a slope of one horizontal to one vertical with a 10 m berm at elevation 1120 m, a 10 m berm at the rock-overburden contact and a slope of three horizontal to one vertical in the overburden above.

The effect of a lower effective angle of shearing resistance of the seam has been examined with a sensitivity analysis of the relationship between the factor of safety and the effective angle of shearing resistance of the seam. Plate D-15 shows the results of this analysis.

Spillway Chute

The spillway chute side slopes have been analyzed by considering three sections along the chute. The location of the phreatic surface and stratigraphy are different in each section. Plates D-16 through D-21 illustrate the sections analyzed and the results obtained. In all cases analysed the steady state seepage case governed.

The analysis dictates that chute side slopes will be stable if they are excavated to one horizontal to one vertical in the rock, with a 10 metre-wide berm every 20 metres height from the base of the excavation, with a 10 metre-wide berm at the rock-overburden contact, and a slope of three horizontal to one vertical in the overburden above.

Plate D-16 illustrates a section approximately 50 m downstream of the centerline of the spillway headworks. The critical sliding surface passes through seam R16 at the elevation 1105 m.

Plate D-18 illustrates a section approximately 140 m downstream of the centerline of the spillway headworks. A sliding surface at the base of the excavation that passes through the claystone and the shear strength along the beddinf for the claystone given in Table D-3 have been used for this sliding surface. Seam R9 is present at elevation 1088 m with an effective angle of shearing resistance of 11.5^{o}. This seam at elevation 1088 m controls the stability, with the factor of safety against failure equal to the required minimum factor of safety.

Plate D-20 illustrates a section approximately 230 m downstream of the spillway headworks. The effect of seam R9 at elevation 1088 m with an effective angle of shearing resistance of 11.5^{o} and seam R1 at the base of the excavation at elevation 1068 m with an effective angle of shearing resistance of 11.5^{o}, were analyzed. In this case the most critical sliding surface passes through the seam at the base of the excavation. The factor of safety along this sliding plane is equal to the required minimum factor of safety.

A sensitivity analysis was done on all three sections to investigate the effect on the factor of safety of the seam having a shear strength lower than its design value. Plates D-17, D-19 and D-21 show the factors of safety for seams at various elevations.

For the slopes in the region of 50 m downstream of the spillway headworks, the calculated safety factor based on a two dimensional analyses is about seven percent less than the required minimum. The design is considered acceptable for the following reasons:

a) The spillway chute is inclined at a slope of six horizontal to one vertical along its axis and the base of the excavated slope. The seams are essentially horizontal and any slide through them will terminate at the upstream end where the seam lies below the bottom of the excavation. As ground surface also slopes downwards in the same direction as the spillway chute excavation, the seam effectively daylights downstream. These constraints will limit the lateral extent of any slide and will cause the actual safety factor to be greater than the calculated.

b) The cohesion present in the mudrock will also cause a slide to be restrained laterally (Baligh and Azzouz (1975)).

c) The phreatic surface assumed in the analyses is high if the combined effect of the grout curtain, the line of pressure relief drains, the drainage blanket beneath the chute slab and river slope on the back side of the excavated slope are taken into account.

D.2 REFERENCES

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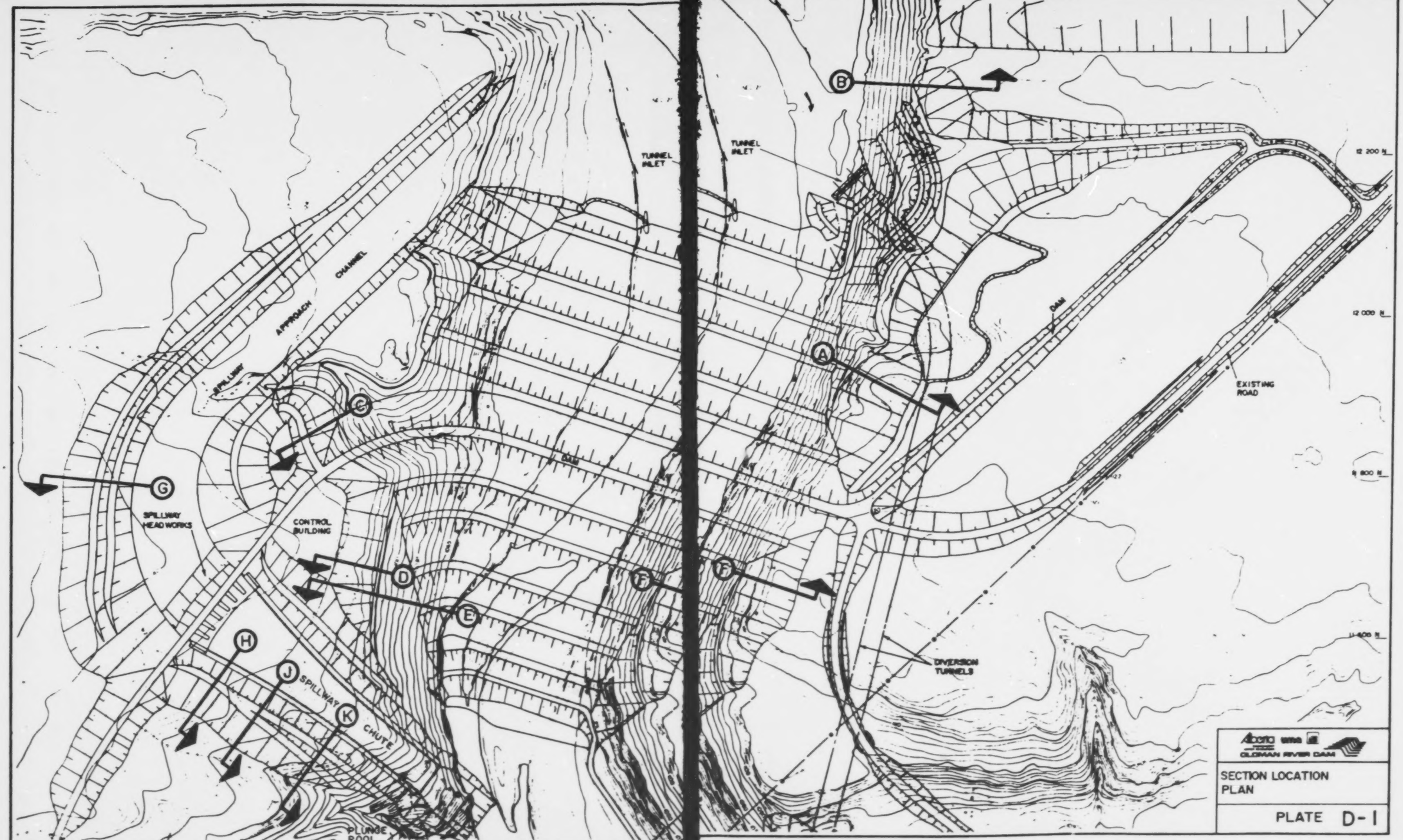
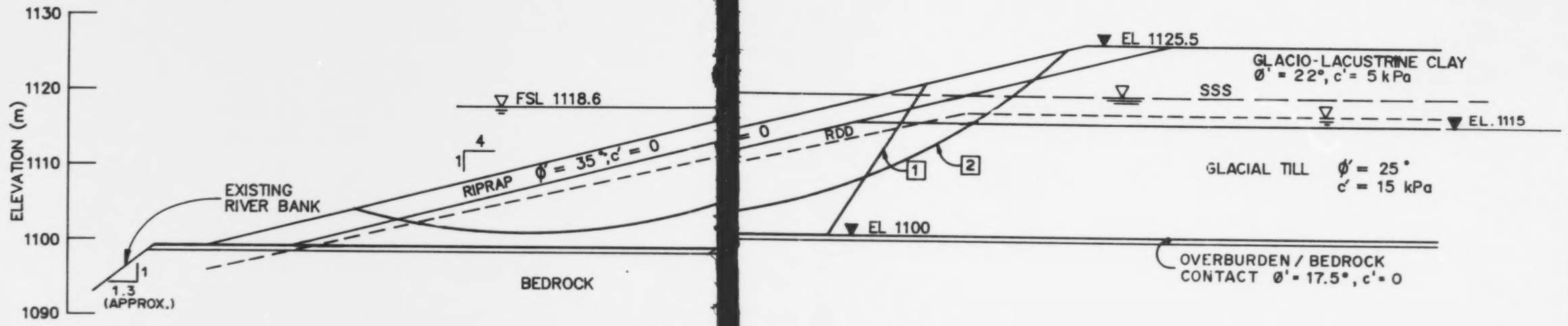

TUNNEL INLET
APPROACH CHANNEL
SPILLWAY
G
SPILLWAY HEADWORKS
C
CONTROL BUILDING
D
E
F
H
J
SPILLWAY CHUTE
K
PLUNGE POOL
B
A
DAM
EXISTING ROAD
12 200 N
12 000 N
11 800 N
DIVERSION TUNNELS
Alcan
OLDMAN RIVER DAM
SECTION LOCATION PLAN
PLATE D-1



SECTION A

SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	COMPOSITE FAILURE		CIRCULAR FAILURE	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS	1.5	1.6	1	2.3	2
2. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL	1.3	1.3	1	1.9	2
3. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL PLUS PSEUDO STATIC SEISMIC LOADING 0.065 g	1.0	1.0	1	1.5	2

LEGEND:

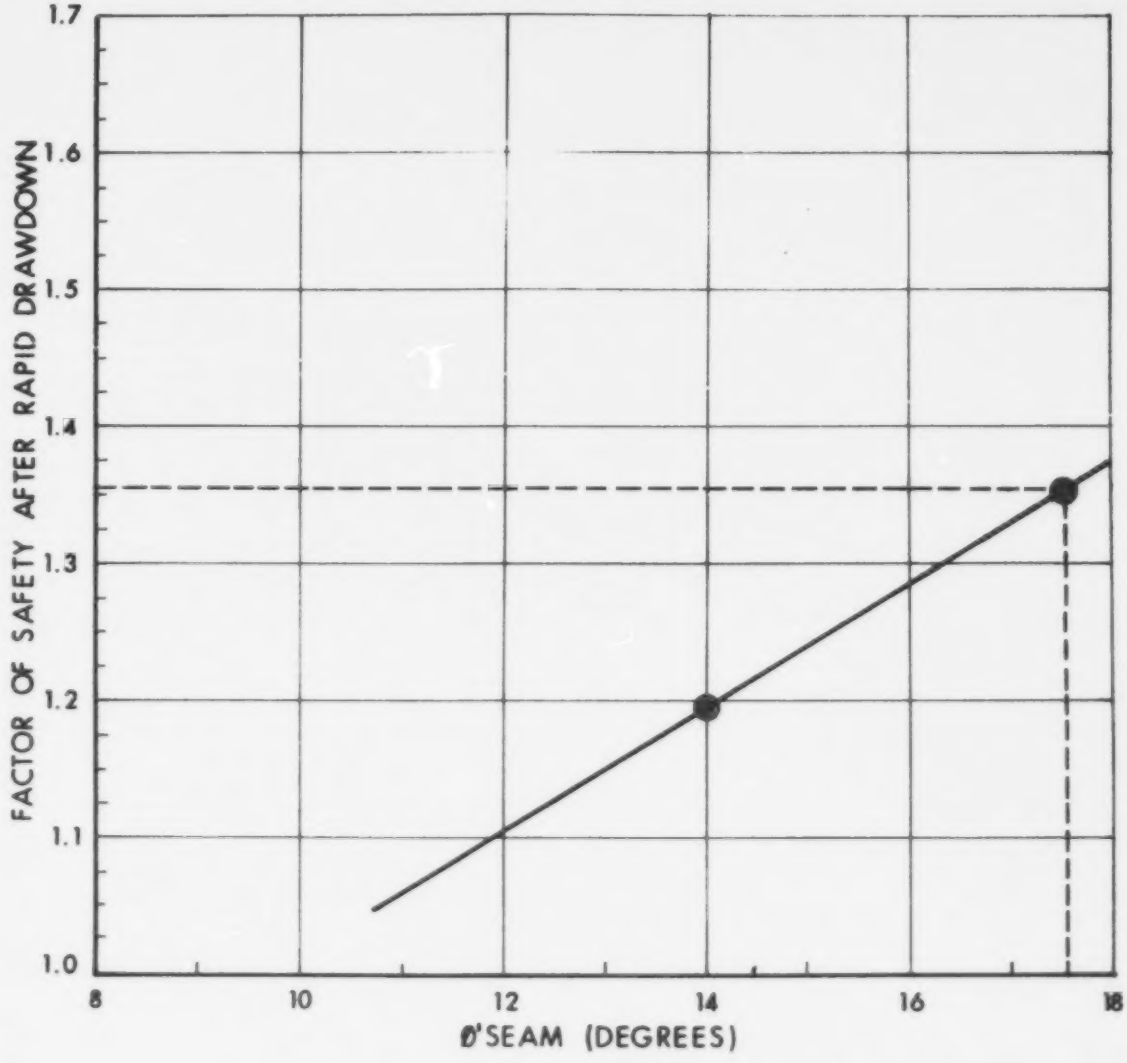
SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

RDD – ASSUMED PHREATIC SURFACE DURING RAPID DRAWDOWN

OLDMAN RIVER DAM

EXCAVATED SOIL SLOPES
UPSTREAM OF DAM
LEFT BANK

PLATE D-2



SECTION A
FACTOR OF SAFETY VS Ø' SEAM

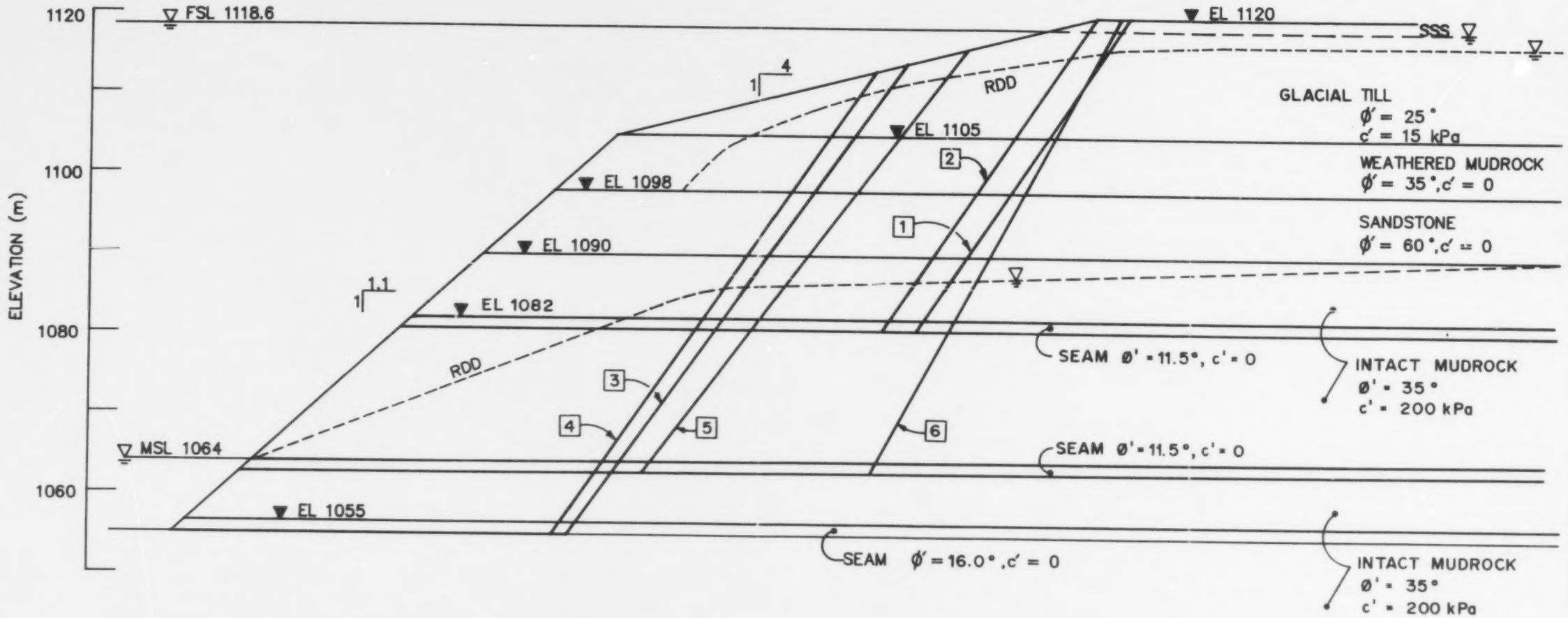
FAILURE THROUGH SEAM AT ELEVATION:

● – 1100 m

Alcan
OLDMAN RIVER DAM

FACTOR OF SAFETY VS Ø' SOIL SLOPES UPSTREAM OF DAM – LEFT BANK

PLATE D-3



CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL. 1055m		SEAM AT EL. 1064m		SEAM AT EL. 1082m	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS	1.5	1.8	3	1.8	5	1.9	1
2. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL	1.3	1.2	3	1.2	5	1.7	2
3. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL PLUS PSEUDO STATIC SEISMIC LOADING 0.065 g	1.0	1.0	4	1.0	6	1.2	1

SECTION B

SCALE 1:500

LEGEND:

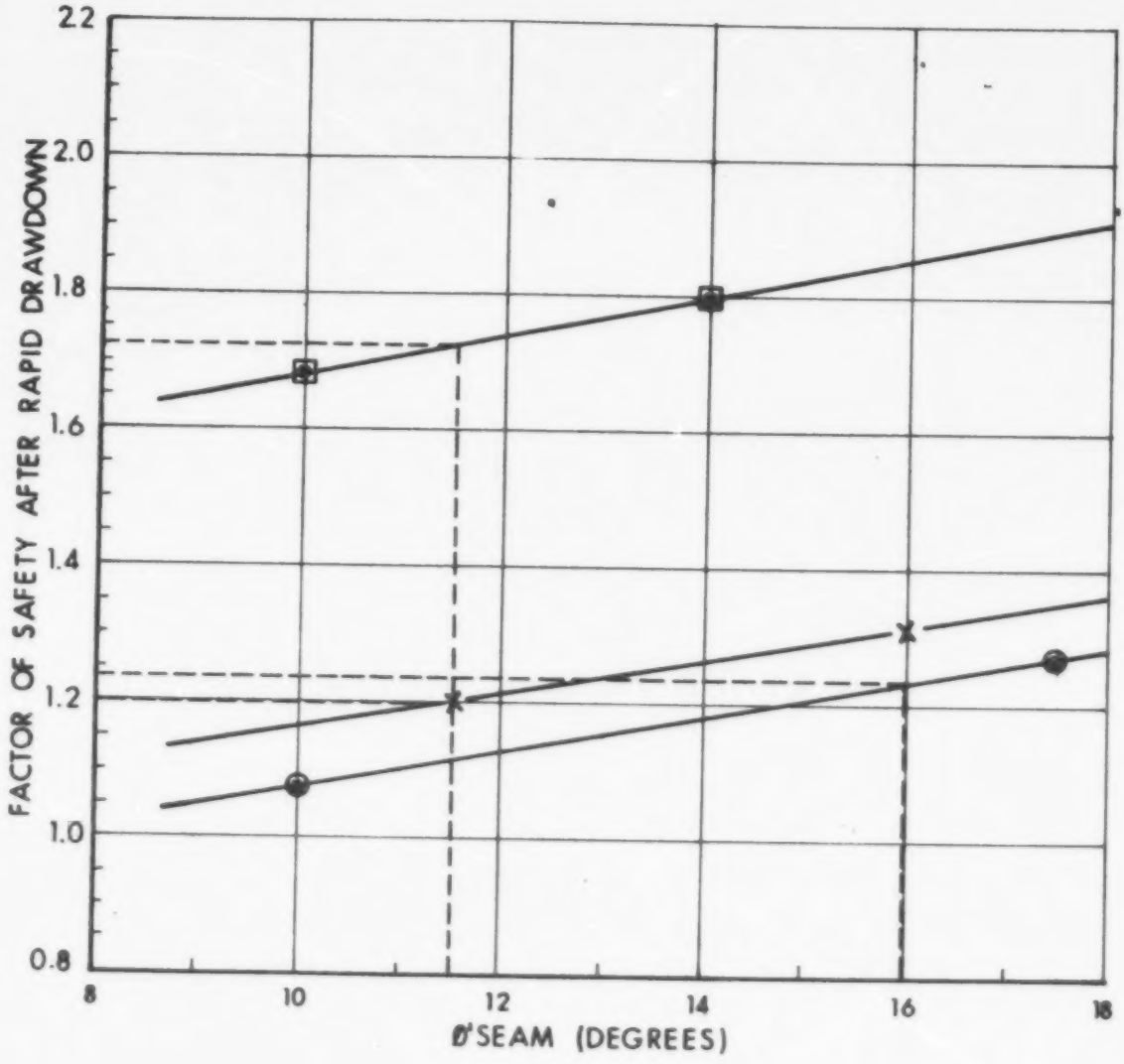
SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

RDD – ASSUMED PHREATIC SURFACE DURING RAPID DRAWDOWN

Alberta Power Limited
DUNKIRK RIVER DAM

EXISTING ROCK SLOPES
UPSTREAM OF DAM
LEFT BANK

PLATE D-4



SECTION B

FACTOR OF SAFETY VS Ø' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

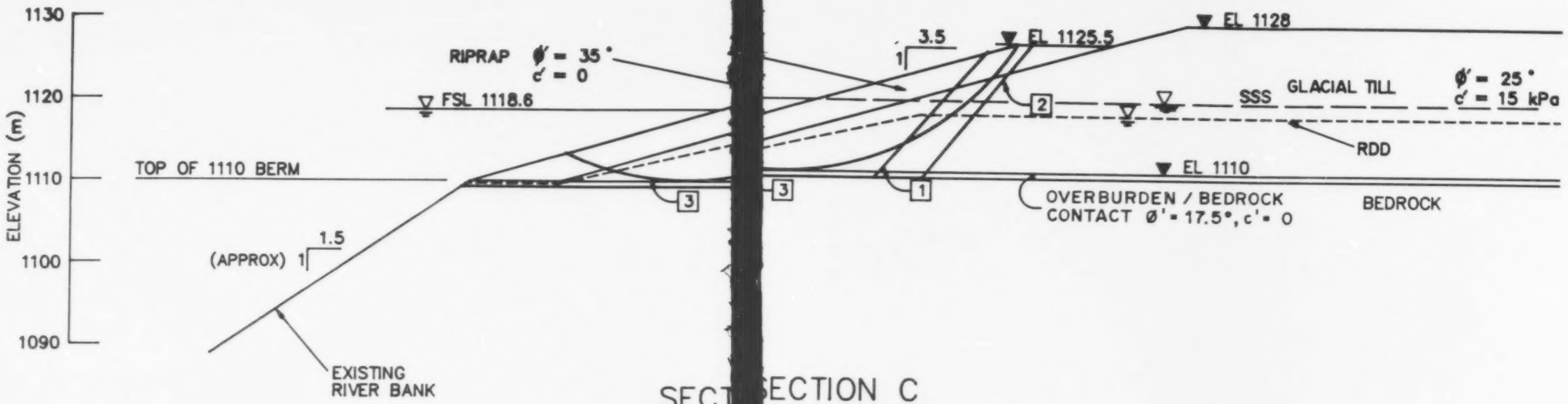
- ● - 1055m
- X - 1064m
- ▣ - 1082m

Alcoa Inc.

GLOOMAN RIVER DAM

FACTOR OF SAFETY VS ø' ROCK SLOPES UPSTREAM OF DAM-LEFT BANK

PLATE D-5



SECTION C

SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	COMPOSITE FAILURE		CIRCULAR FAILURE	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS	1.5	1.5	1	2.0	3
2. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL	1.3	1.4	2	2.0	3
3. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL PLUS PSEUDO STATIC SEISMIC LOADING 0.065 g	1.0	1.1	2	1.6	3

LEGEND:

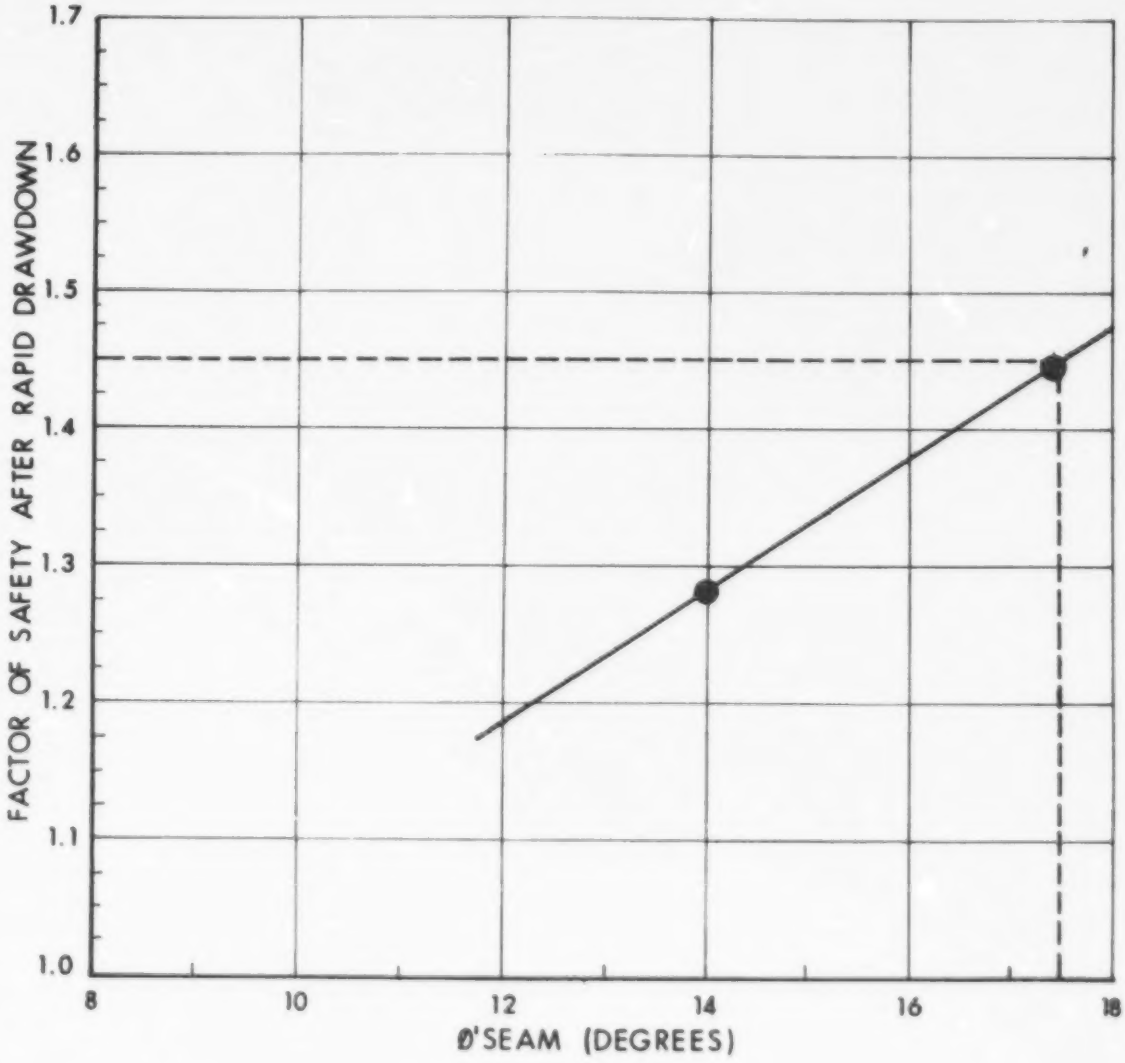
SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

RDD – ASSUMED PHREATIC SURFACE DURING RAPID DRAWDOWN

Alberta Power Limited
OLDMAN RIVER DAM

EXCAVATED SOIL SLOPES
UPSTREAM OF DAM
RIGHT BANK

PLATE D-6



SECTION C

FACTOR OF SAFETY VS Ø' SEAM

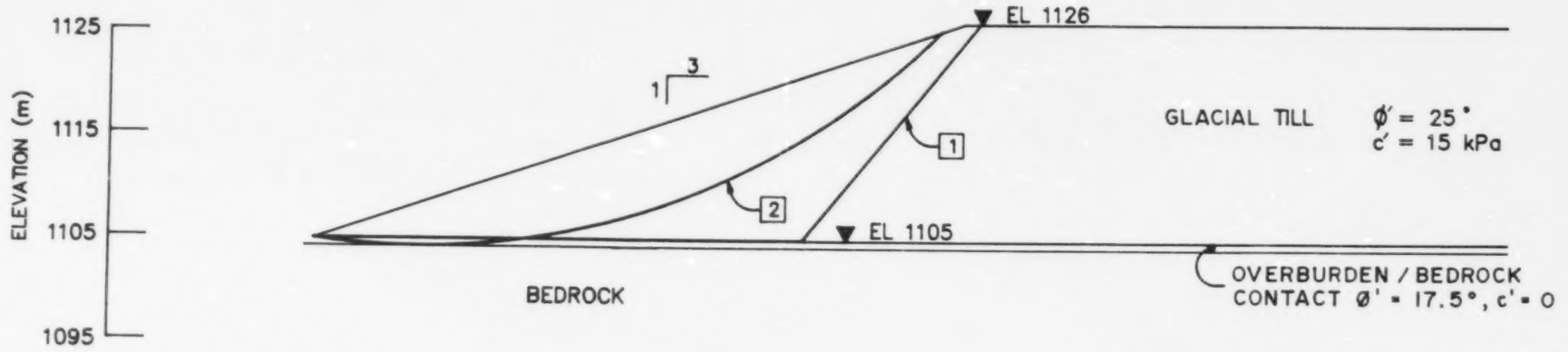
FAILURE THROUGH SEAM AT ELEVATION:

● – 1110 m

OLDMAN RIVER DAM

FACTOR OF SAFETY VS Ø' SOIL SLOPES UPSTREAM OF DAM – RIGHT BANK

PLATE D-7



SECTION D

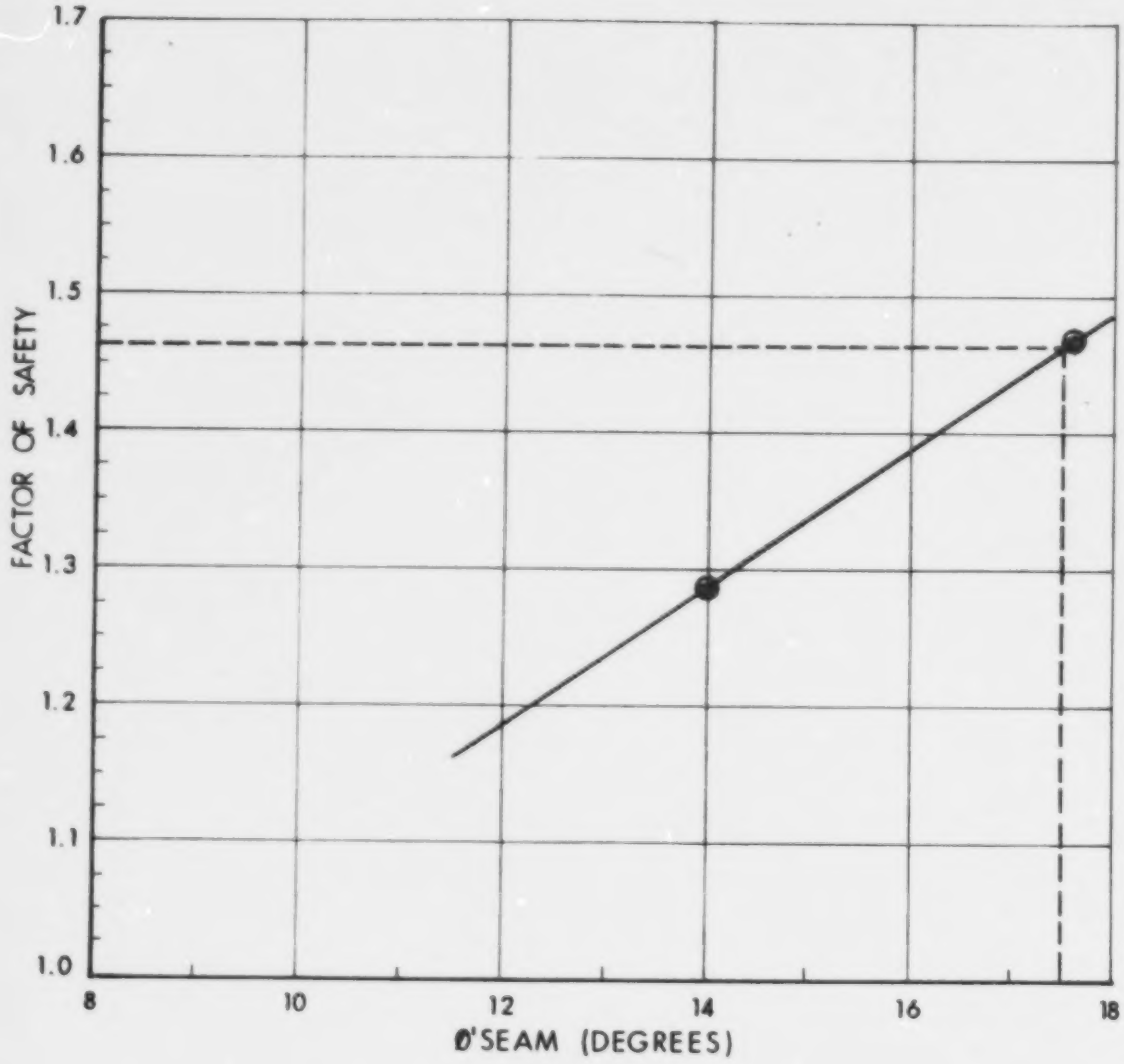
SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	COMPOSITE FAILURE		CIRCULAR FAILURE	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS	1.5	1.8	1	2	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH PSEUDO-STATIC SEISMIC LOADING 0.065 g	1.0	1.3	1	1.6	2

OLDMAN RIVER DAM

EXCAVATED SOIL SLOPES
DOWNSTREAM OF DAM
RIGHT BANK

PLATE D-8



SECTION D

FACTOR OF SAFETY VS Ø' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

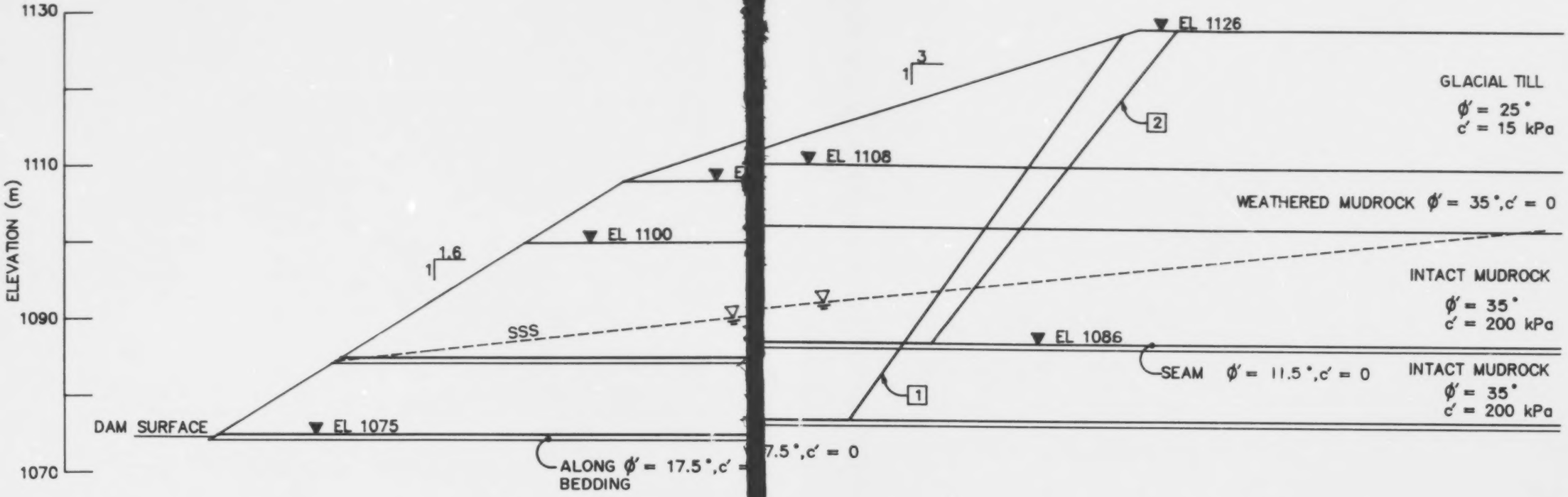
● - 1105m

Alberta Utilities Commission

OLDMAN RIVER DAM

FACTOR OF SAFETY VS Ø' SOIL SLOPES DOWNSTREAM OF DAM-RIGHT BANK

PLATE D-9



SECTION E

SCALE 1:1000

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL. 1075 m		SEAM AT EL. 1086m	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS	1.5	1.5	1	1.5	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS PLUS PSEUDO-STATIC SEISMIC LOADING 0.065 g	1.0	1.3	1	1.2	2

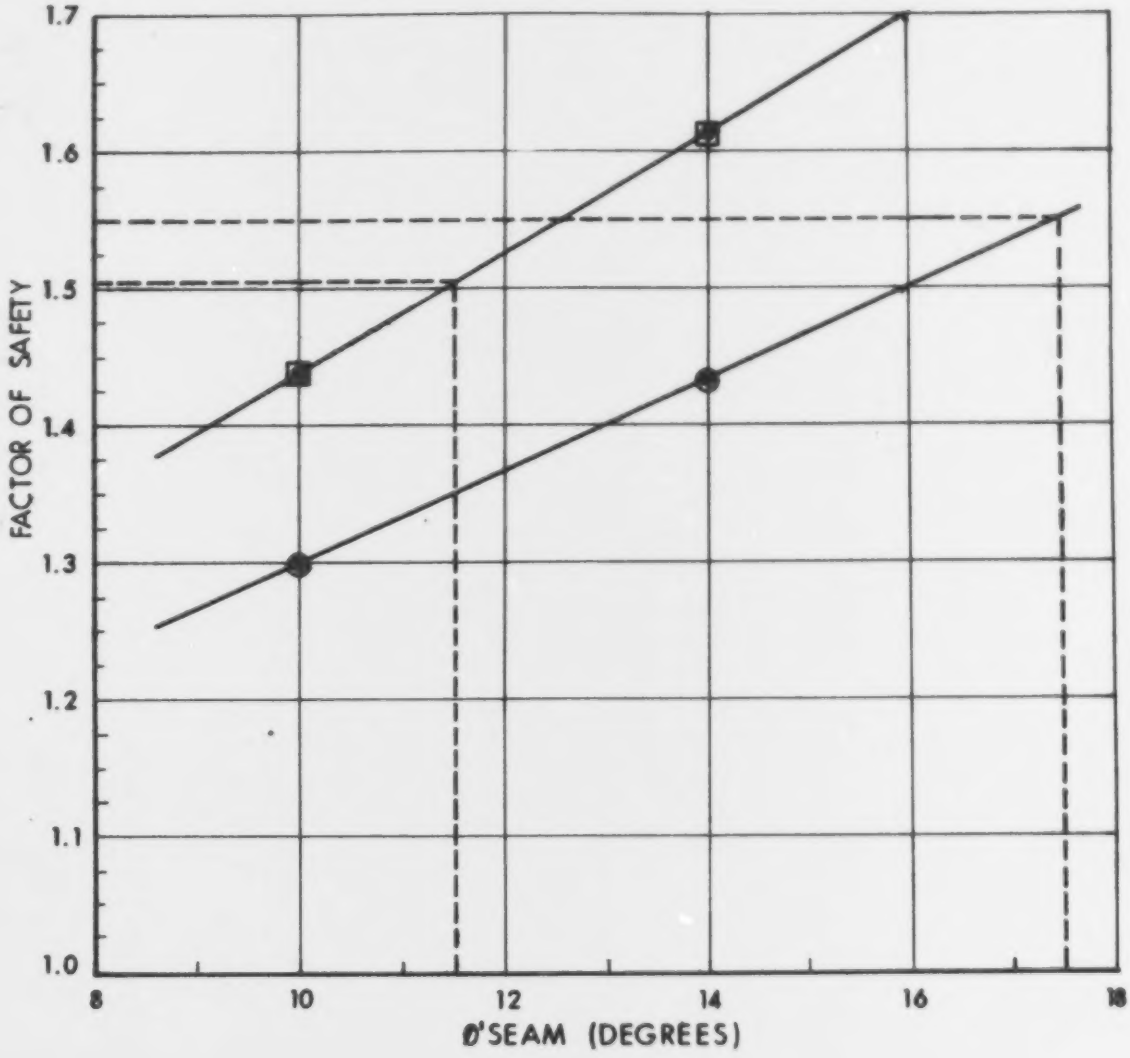
LEGEND:

SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

OLDMAN RIVER DAM

EXISTING ROCK SLOPES
DOWNSTREAM OF DAM
RIGHT BANK

PLATE D-10



SECTION E

FACTOR OF SAFETY VS Ø' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

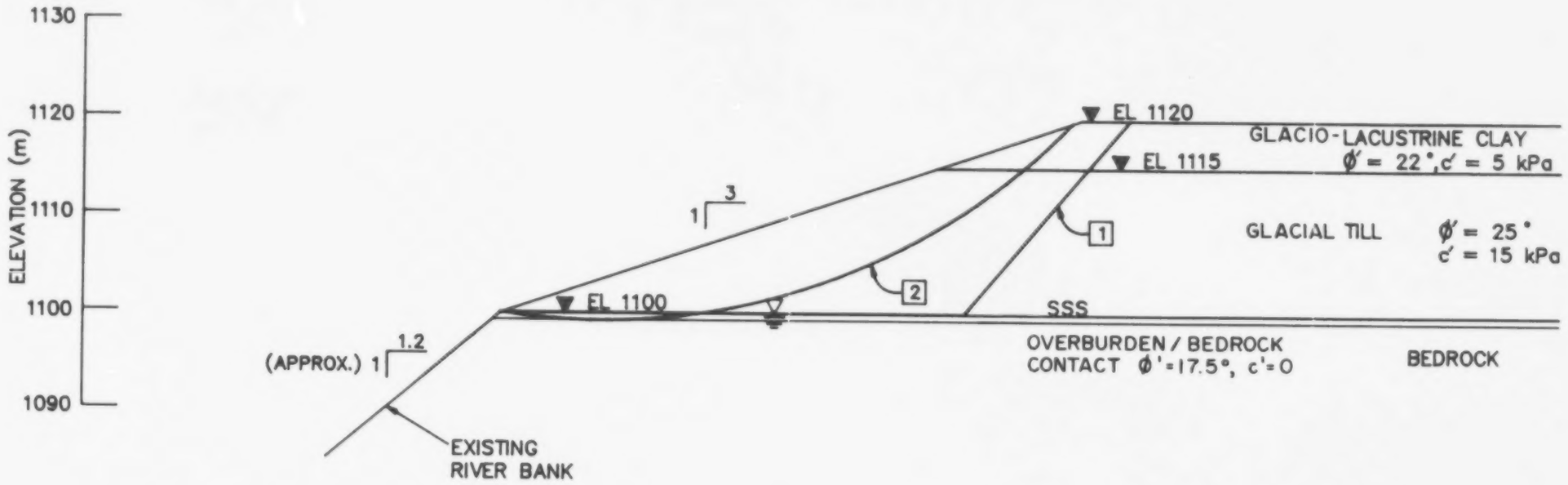
● - 1075 m

⊡ - 1086 m

CLARKSON RIVER DAM

FACTOR OF SAFETY VS Ø' ROCK SLOPES DOWNSTREAM OF DAM - RIGHT BANK

PLATE D-11



SECTION F

SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	COMPOSITE FAILURE		CIRCULAR FAILURE	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS	1.5	1.5	1	1.9	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH PSEUDO-STATIC SEISMIC LOADING 0.065 g	1.0	1.2	1	1.5	2

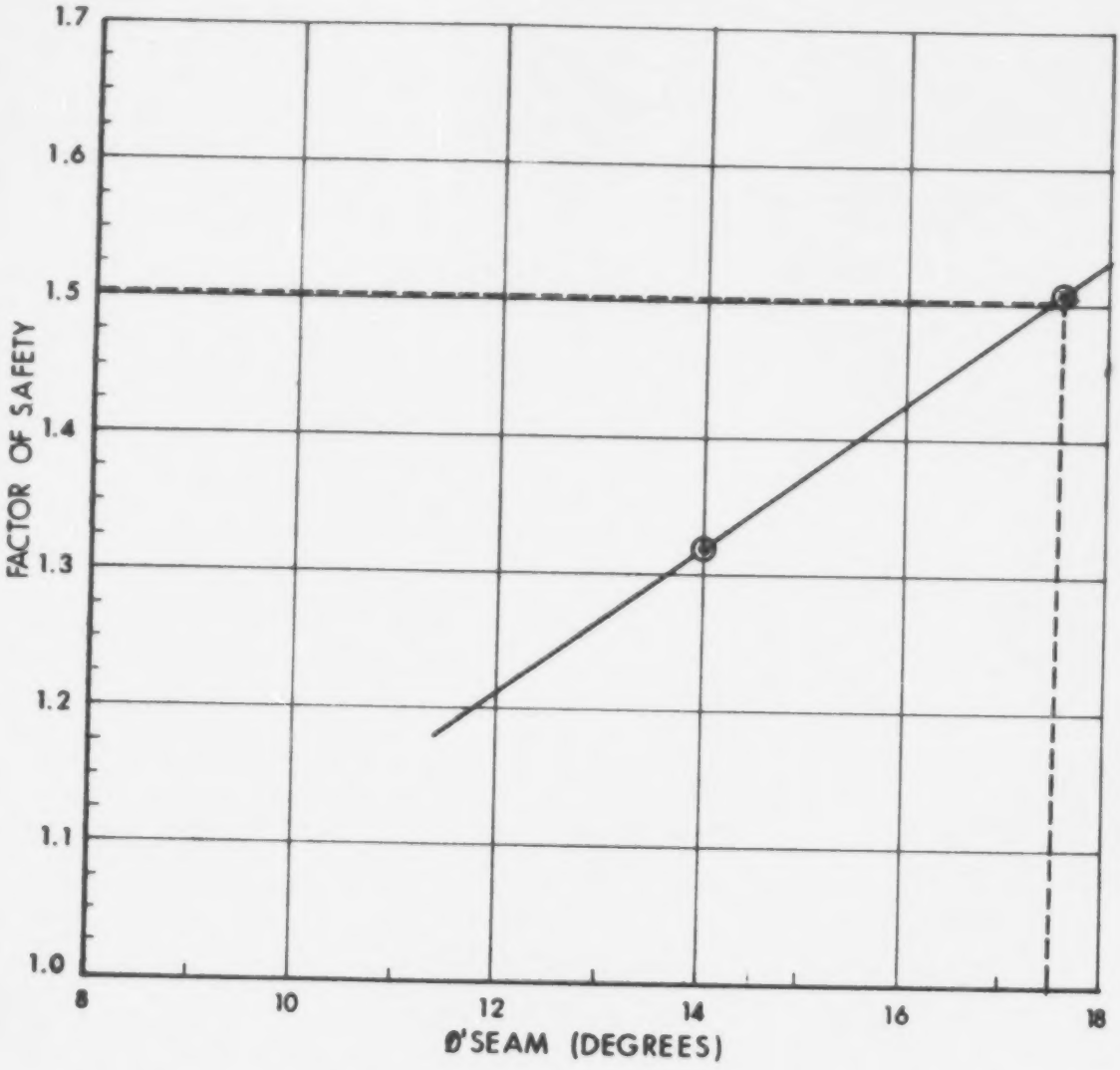
LEGEND:

SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

Alcan inc.
OLDMAN RIVER DAM

EXCAVATED SOIL SLOPES
DOWNSTREAM OF DAM
LEFT BANK

PLATE D-12



SECTION F

FACTOR OF SAFETY VS Ø' SEAM

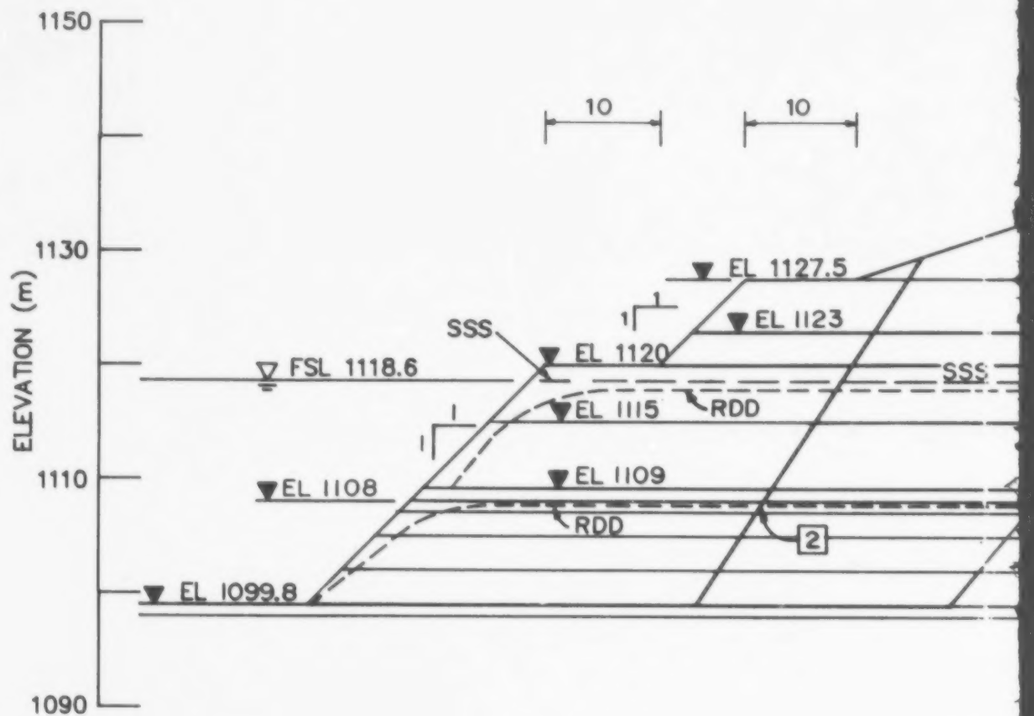
FAILURE THROUGH SEAM AT ELEVATION:

● - 1100m

Alcan
OLDMAN RIVER DAM

FACTOR OF SAFETY VS ø'
SOIL SLOPES DOWNSTREAM
OF DAM - LEFT BANK

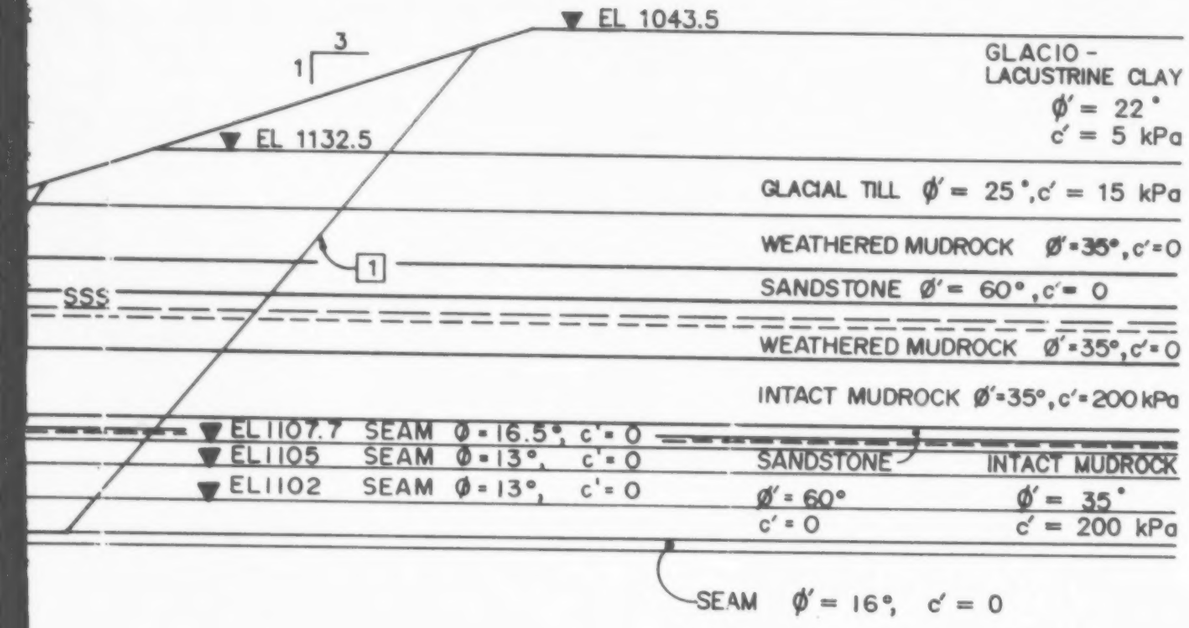
PLATE D-13



SEC

SCALE

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL 1099.8m	
		F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS	1.5	1.8	1
2. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL	1.3	1.6	2
3. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL PLUS PSEUDO-STATIC SEISMIC LOADING 0.065 g	1.0	1.4	2



GLACIO - LACUSTRINE CLAY
$\phi' = 22°$
$c' = 5$ kPa

GLACIAL TILL $\phi' = 25°, c' = 15$ kPa

WEATHERED MUDROCK $\phi' = 35°, c' = 0$

SANDSTONE $\phi' = 60°, c' = 0$

WEATHERED MUDROCK $\phi' = 35°, c' = 0$

INTACT MUDROCK $\phi' = 35°, c' = 200$ kPa

SANDSTONE	INTACT MUDROCK
$\phi' = 60°$	$\phi' = 35°$
$c' = 0$	$c' = 200$ kPa

SECTION G

SCALE 1:500

LEGEND:

SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

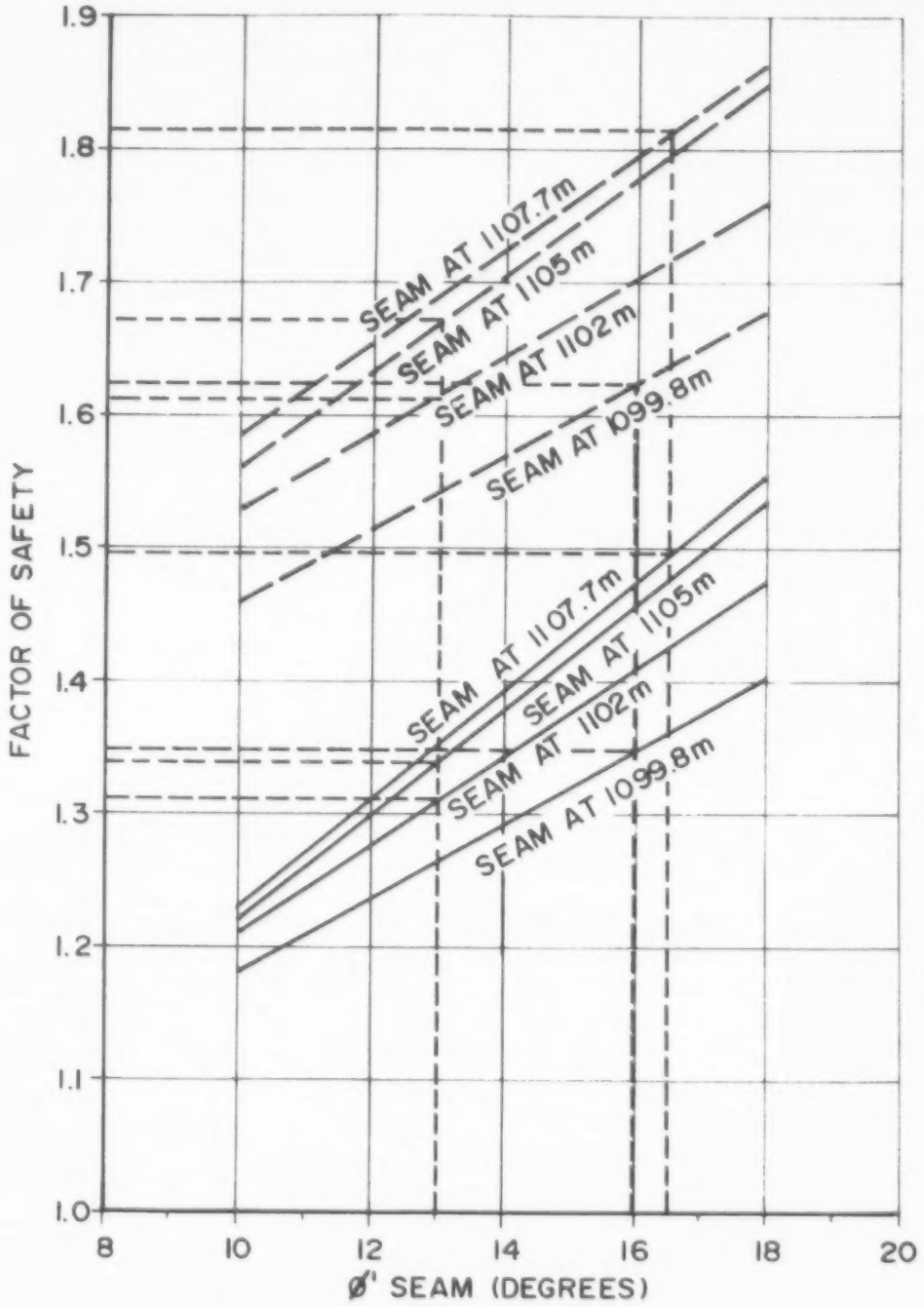
RDD – ASSUMED PHREATIC SURFACE DURING RAPID DRAWDOWN

SSS – PHREATIC SURFACE DURING STEADY STATE CONDITIONS

Alberta Power Limited
OLDMAN RIVER DAM

APPROACH CHANNEL
SIDE SLOPES STABILITY

PLATE D-14



SECTION G
APPROACH CHANNEL
SIDE SLOPE STABILITY
RAPID DRAWDOWN

LEGEND

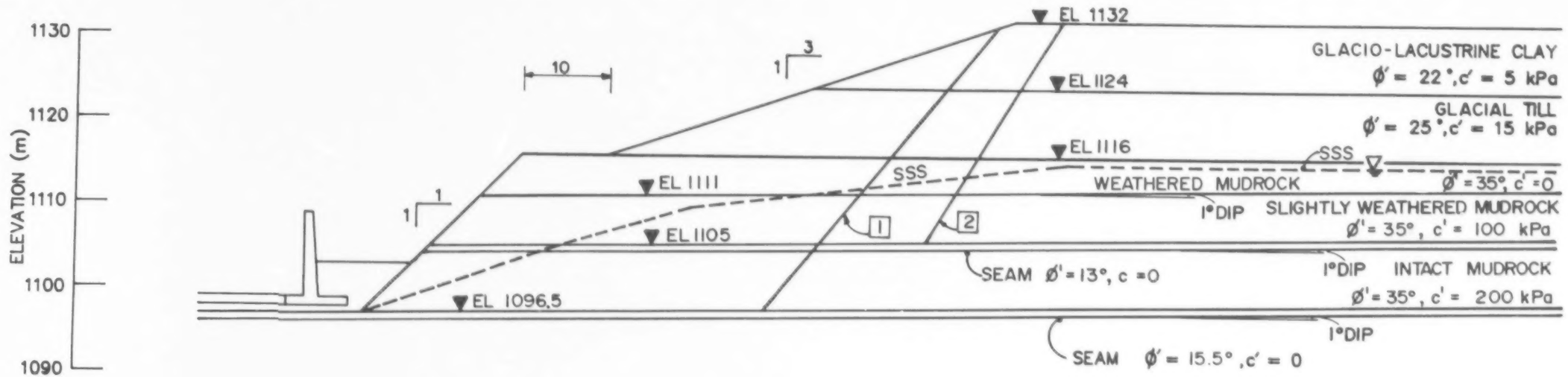
— — — WITHOUT EARTHQUAKE

——— WITH EARTHQUAKE (0.065 g)

Alberta
OLDMAN RIVER DAM

FACTOR OF SAFETY vs Ø'
STABILITY OF APPROACH
CHANNEL SIDE SLOPES

PLATE D-15



SECTION H

SCALE 1:500

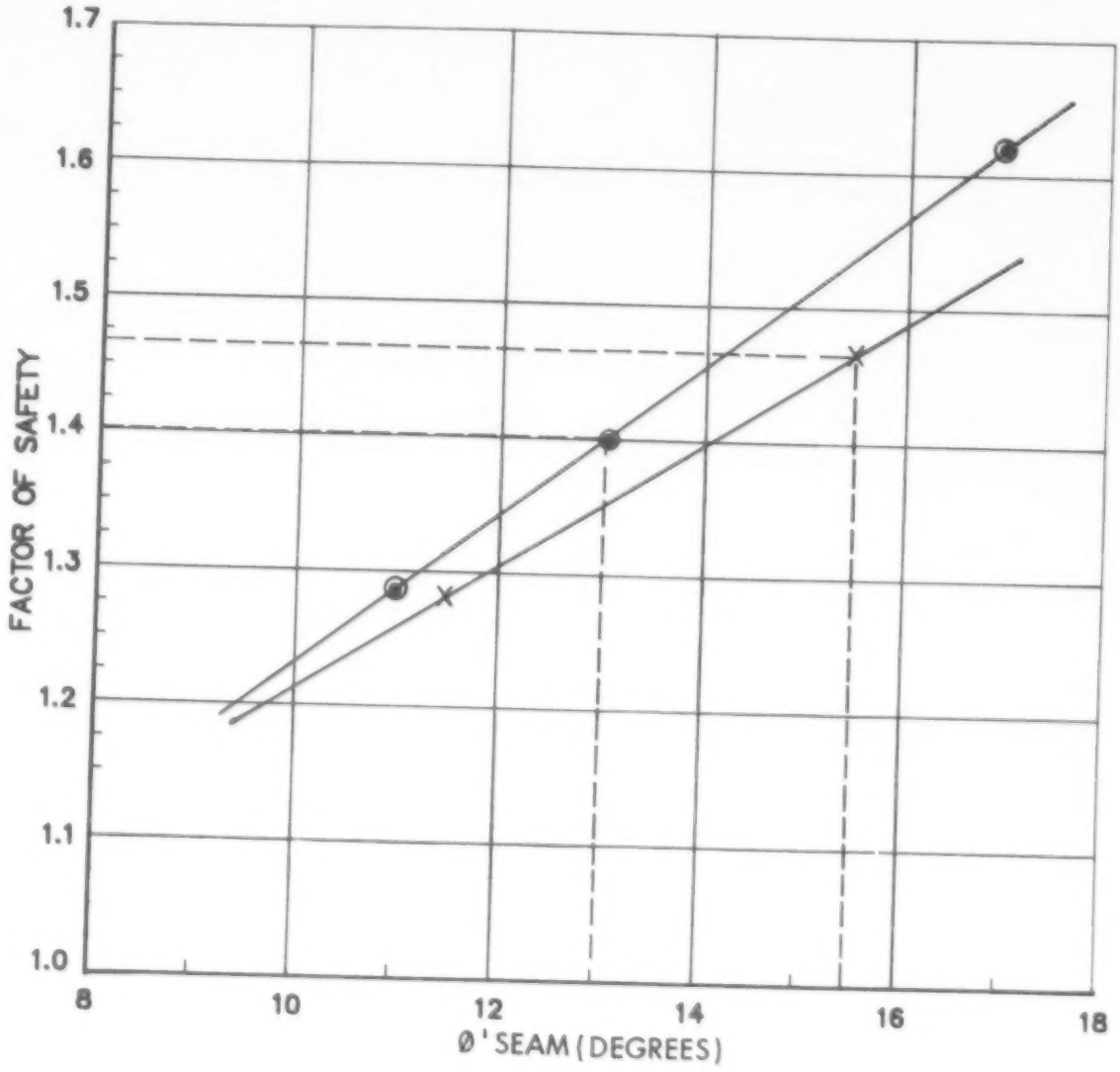
CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL. 1096.5m		SEAM AT EL. 1105m	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE	1.5	1.5	1	1.4	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS AND PSEUDO-STATIC SEISMIC LOADING 0.065g	1.0	1.1	1	1.1	2

LEGEND:
SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

OLDMAN RIVER DAM

SPILLWAY CHUTE STABILITY
SIDE SLOPES EXCAVATION

PLATE D-16



SECTION H

FACTOR OF SAFETY VS ϕ' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

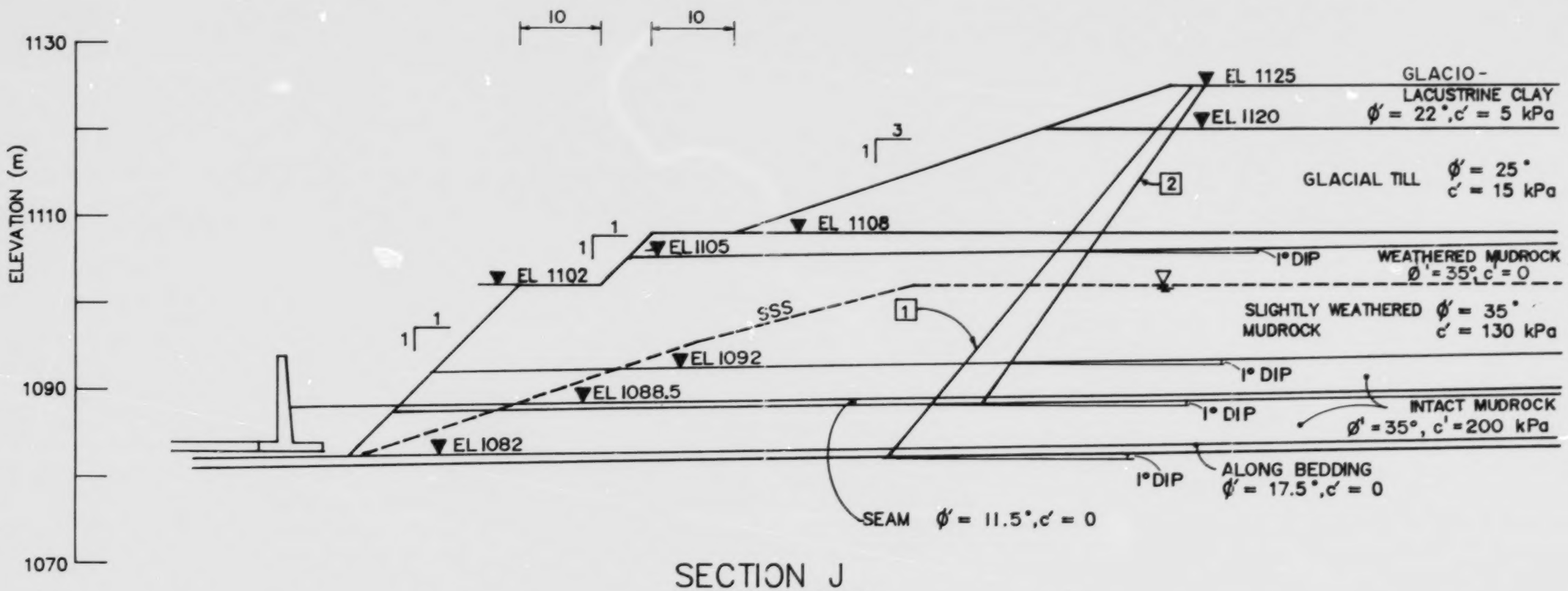
× – 1096.5 m

● – 1105 m

Alberta
OLDMAN RIVER DAM

FACTOR OF SAFETY VS ϕ'
STABILITY OF SPILLWAY
CHUTE SIDE SLOPES

PLATE D-17



SECTION J

SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL. 1082 m		SEAM AT EL 1088.5 m	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS	1.5	1.6	1	1.5	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS PLUS PSEUDO-STATIC SEISMIC LOADING 0.065g	1.0	1.3	1	1.1	2

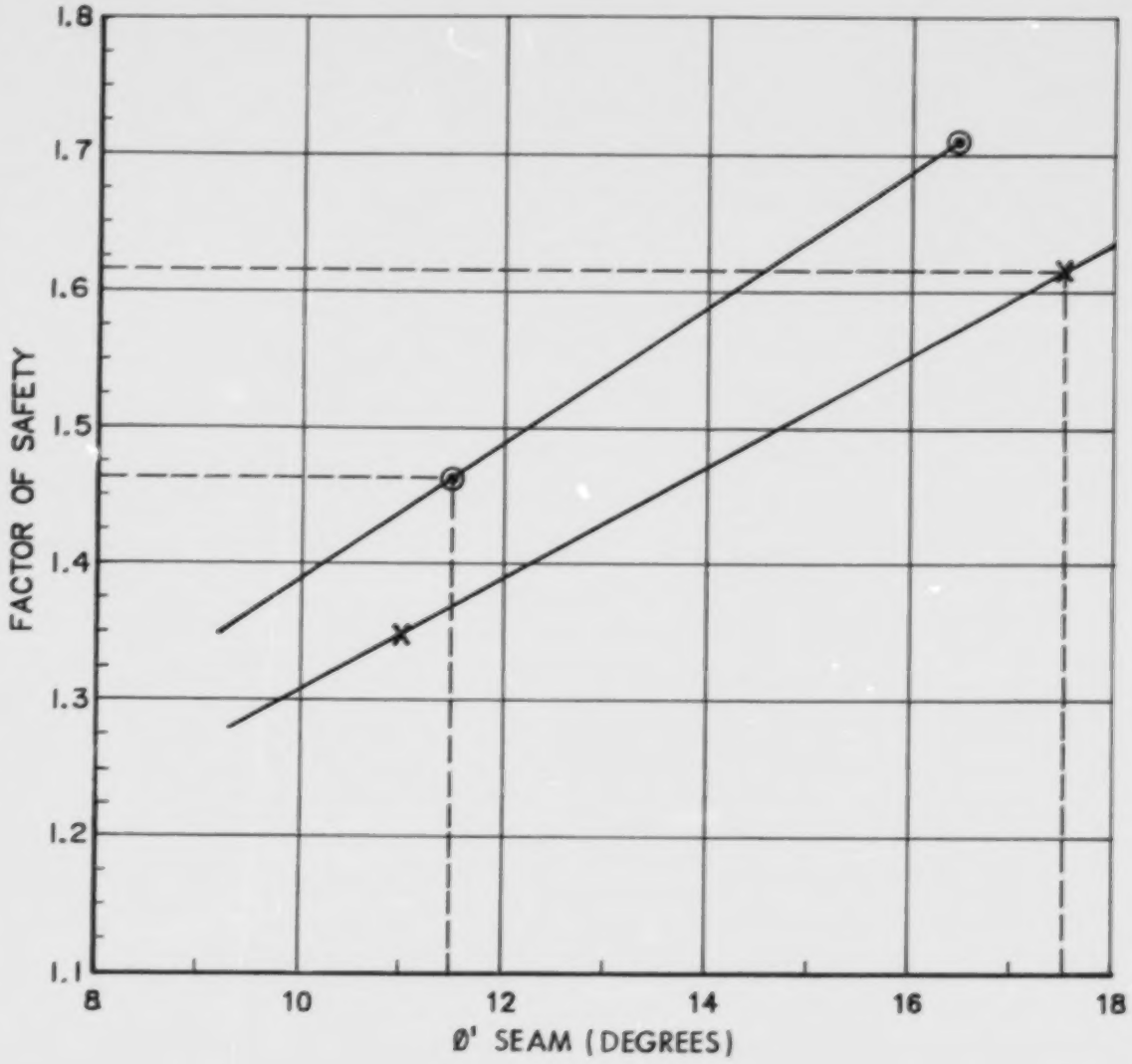
LEGEND:

SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

Alberta Power Limited
OLDMAN RIVER DAM

SPILLWAY CHUTE STABILITY
SIDE SLOPES EXCAVATION

PLATE D-18



SECTION J

FACTOR OF SAFETY VS ϕ' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

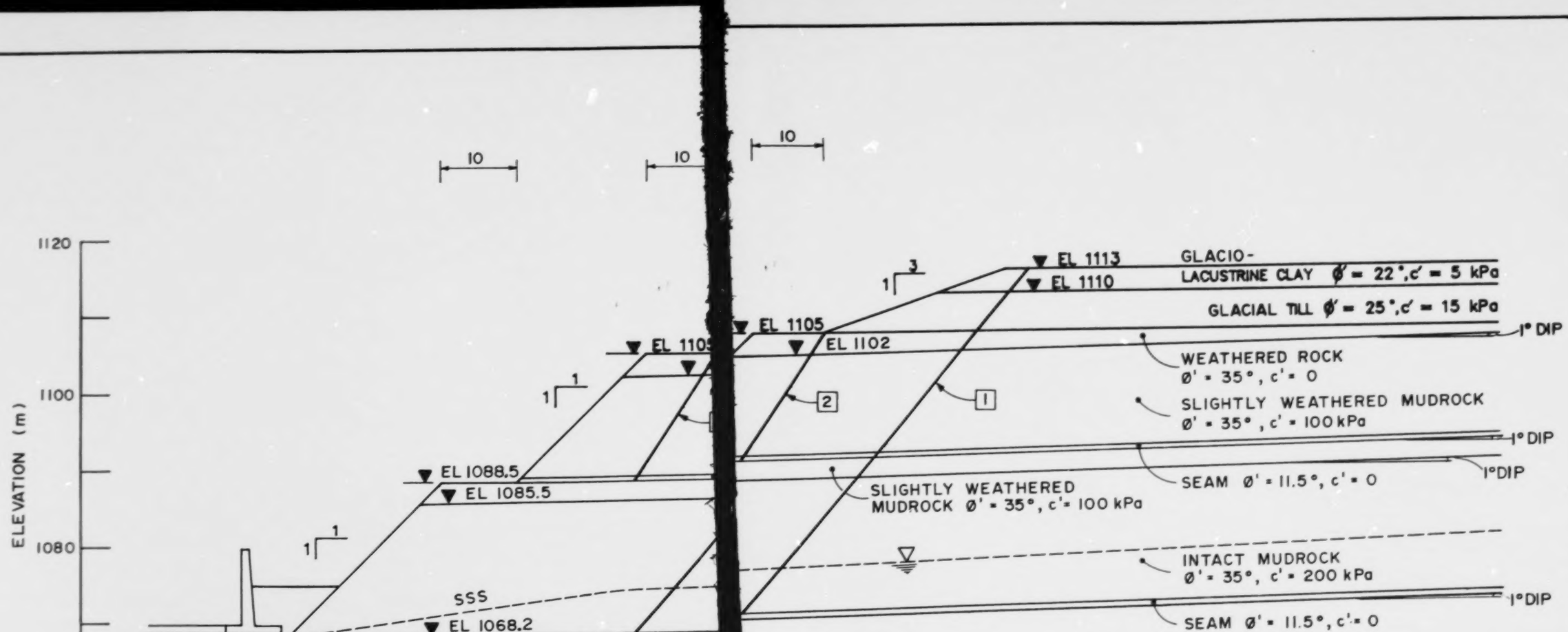
× – 1082 m

● – 1088.5 m

GLENMAN RIVER DAM

FACTOR OF SAFETY VS ϕ'
STABILITY OF SPILLWAY
CHUTE SIDE SLOPES

PLATE D-19



SECTIO

SCALE 1:50

ECTION K

CALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL. 1068.2 m		SEAM AT EL. 1088.5 m	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS	1.5	1.5	1	1.7	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS PLUS PSEUDO-STATIC SEISMIC LOADING 0.065g	1.0	1.2	1	1.4	2

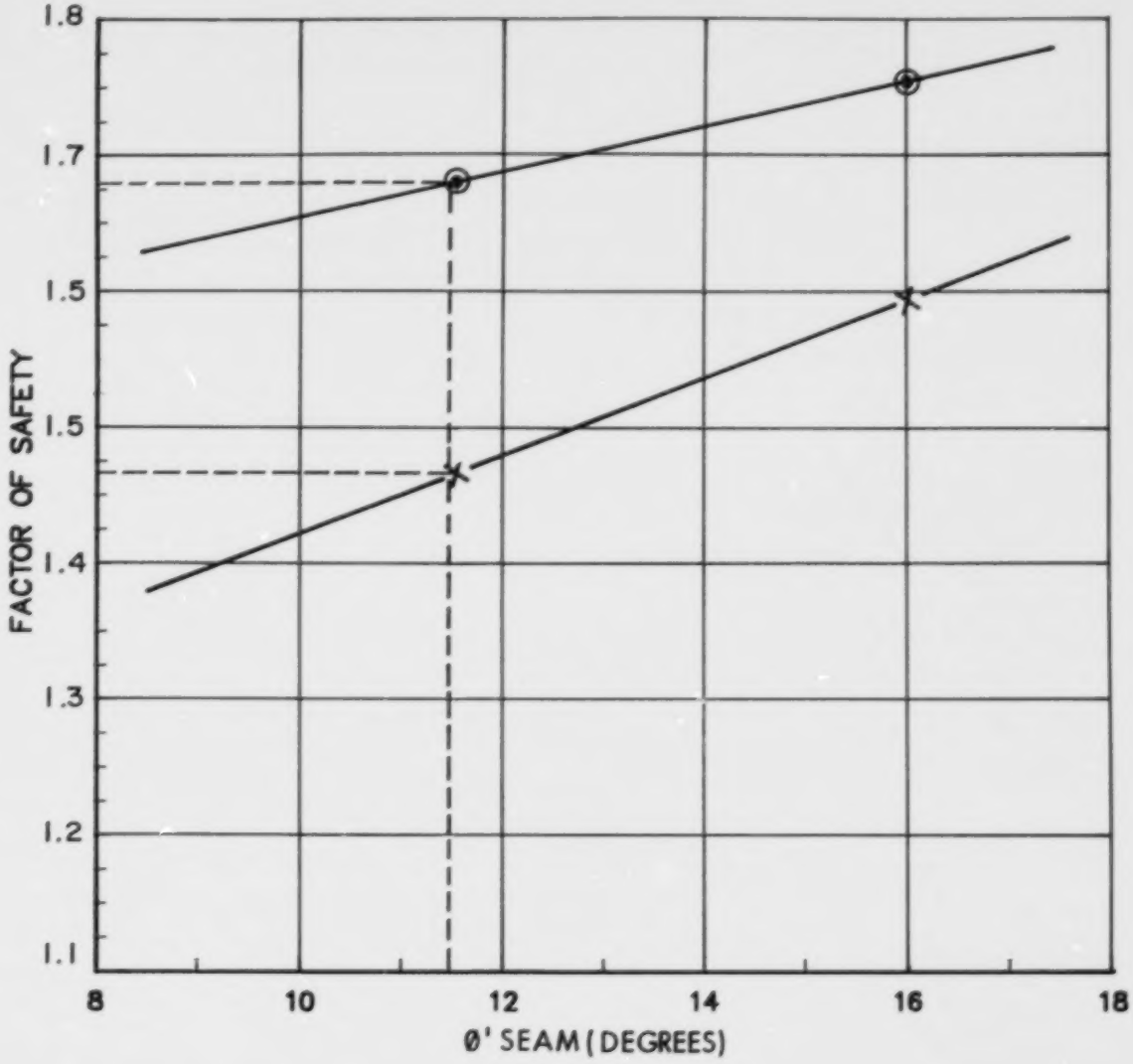
8.5 m
RFACE

LEGEND:
SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

Alberta Power
OLDMAN RIVER DAM

SPILLWAY CHUTE STABILITY
SIDE SLOPES EXCAVATION

PLATE D-20



SECTION K

FACTOR OF SAFETY VS Ø' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

◉ – 1088.5m

× – 1068.2m

Alberta Power (1981) Limited
CLEARWATER RIVER DAM

FACTOR OF SAFETY VS Ø'
STABILITY OF SPILLWAY
CHUTE SIDE SLOPES

PLATE D-21

OLDMAN RIVER DAM

DESIGN REPORT

APPENDIX E

SEEPAGE ANALYSIS

September, 1988
Updated June, 1989

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E-15 LEFT RIVER BANK - SECTION D - DRAINAGE TUNNEL - LOCATION - 2
E-16 LEFT RIVER BANK - SECTION D - DRAINAGE TUNNEL - LOCATION - 3
E-17 LEFT RIVER BANK - SECTION D - DRAINAGE TUNNEL - LOCATION - 4
E-18 LEFT RIVER BANK - SECTION D - DRAINAGE TUNNEL - LOCATION - 5
E-19 LEFT RIVER BANK - SECTION D - DRAINAGE TUNNEL - LOCATION - 6

E-1 INTRODUCTION

Seepage analyses were done primarily to determine the flow patterns, to provide pore pressure input to the stability analyses of structures during steady state seepage conditions, to provide data for use in establishing the effective length and depth of the grout curtain and to check the magnitude of hydraulic gradients within the dam and in the vicinity of the drainage tunnels.

The hydraulic conductivities used in the finite element seepage analyses were derived from insitu permeability testing (pressure packer testing) during the site investigation. Interpretation of these test data were complicated because of the water table which is within a few metres above river level across the entire site except for minor perched water tables. Although each test was preceded by an attempt to saturate the ground in the vicinity of the test section of each drill hole, the degree of saturation achieved could not be assessed. It did however influence the permeability results.

Further complications were found in the interpretation of the very high flows into the rock that were recorded in some of the tests. These could have resulted from either faulty seating of the packer or could have been a function of the amount and size of the joints in the rock. Large open joints were observed in the sandstone and often in the siltstone layers in rock cores, in the shafts and in rock exposures. Joint openings of several tens of millimetres were observed in the shafts; some of which were dry but seepage in others was large. The total seepage in the shaft located in the centre of the river valley was so great that it could not be entered. One other shaft was unpleasantly wet. Many of the joints observed in the shafts and in drillholes appear to be in direct hydraulic contact with the river and and are capable of discharging large volumes of water. It is considered however, that the insitu permeability test results give a good indication of the overall relative permeability of the rock.

Their use in these seepage analyses for the purposes stated above is therefore appropriate. Because the packer test permeabilities are probably not representative of the actual permeability of the rock, particularly if the rock is jointed, and because the finite element program used in the analysis is not capable of modelling discrete, concentrated flows through joints, the seepage analyses have not been used to estimate the quantity of groundwater seepage through, beneath and around the Oldman River Dam. The seepage analyses did show that the grout curtain will reduce losses from the reservoir to a very low quantity. The analyses also indicated that downstream drainage measures incorporated into the design will control the outflow conditions of groundwater.

E-2 OBJECTIVES

The project site was divided into three regions for the purposes of the analyses. These were the left river bank, the right river bank including the spillway excavation and the dam.

The specific objectives of the analysis of each region were as follows:

a.) Left Abutment

The objectives were:

i) to predict the location of the line of intersection between the phreatic surface and the ground surface, and

ii) to optimize extent of the grout curtain and the relative location of the drainage tunnel.

b.) Right Abutment

The objectives were:

i) to predict the location of the line of intersection between the phreatic surface and the ground surface in the area of the spillway chute,

ii) to optimize the extent of the grout curtain, and the relative location of the drainage tunnel, and

iii) to determine the porewater pressures underneath the spillway for use in the stability analyses.

c.) Dam

The objectives were:

i) to adjust the location of the drain zones to reduce the hydraulic gradient at the downstream contact of the core with bedrock, and

ii) to determine the pore pressures in the core during steady state seepage with the reservoir at full supply level for use in the stability analyses.

E-3 METHOD

a) Computer Programs

A finite element computer program called "SEEP" was used. This program was originally written for a main frame computer by Dr. Taylor of the University of California in 1969. It has recently been down-loaded for use on an IBM-PC computer. The program analyses both plane, two dimensional and axisymmetric flow through either confined or unconfined porous media. Anisotropic and heterogeneous media can also be simulated. The program will determine the location of the phreatic surface for the unconfined problem.

The output from SEEP was used as input to another computer program called SEEPLOT, which plots the geometry of each two dimensional section analyzed with the computed equipotential lines.

b) Permeability Parameters

The permeability coefficients used in the analyses for the soil and rock are listed in Table E1. The analyses were also done using a ratio of horizontal to vertical coefficients k_H/k_V = 100.

TABLE E-1

PERMEABILITY COEFFICIENTS

	HORIZONTAL (cm/s)	VERTICAL (cm/s)
Foundation Material		
River Alluvium	1	10^{-1}
Valley bedrock above el. 1030 m	10^{-4}	10^{-5}
Valley bedrock below el. 1030 m	10^{-5}	10^{-6}
Right river bank bedrock above el. 1070 m	10^{-4}	10^{-5}
Left river bank bedrock above el. 1070 m	$5x10^{-3}$	$5x10^{-4}$
Right river bank bedrock below el. 1070 m	10^{-5}	10^{-6}
Left river bank bedrock below el. 1070 m	$5x10^{-5}$	$5x10^{-6}$
Grout Curtain	10^{-6}	10^{-6}
Glaciolacustrine Silt and Clay	10^{-5}	10^{-6}
Compacted Fill Materials		
Zones 1 & 1A (impervious)	10^{-6}	10^{-7}
Zones 2, 3, 5 & 7 (Granular)	10^{-1}	10^{-1}

Note: The permeability coefficients are listed on the plates in the units of m/s that are required as input to the program. The more familiar units of cm/s are given in the table.

c) Three Dimensional Flow Simulation

Three dimensional flow around the dam and through the river banks was modelled by first sketching a rough horizontal flow net. Sections were then drawn parallel to the flow path indicated by the flow net. The sections that were analyzed are shown on Plate E-1. Two dimensional seepage flow through each section was then determined using SEEP and the results were integrated manually.

d) Results

The results of the analysis are presented on Plates E-2 to E-19 and are discussed in the following sections. The permeability parameters used are based on interpreted in situ drillhole packer test results from the site investigation.

E-4 LEFT RIVER BANK

Seepage through the left bank was analyzed for a distance of 400 m from the valley wall. Four sections were drawn for the two-dimensional analysis. First a base case without the grout curtain and the drainage tunnel was analyzed for all four sections. The effect of the grout curtain was then simulated by introducing a narrow vertical zone with a lower horizontal permeability from the surface to elevation 1070. All other values were left unchanged. The presence of the diversion tunnels was not modelled.

The effect of the grout curtain is demonstrated on Plate E-2 which shows the reduction in seepage discharge for different assumed permeabilites for the grout curtain, for each of the four sections analyzed. Section D is closest to the river bank and section A is the furthest away as shown on Plate E-1. Plate E-2 also shows the decreasing effectiveness of the grout curtain as the grout curtain is extended into the river bank. The equipotential plots for the flow sections analyzed are presented on Plates E-3 to E-6 respectively.

It was concluded from the analysis that the proposed grout curtain from the left abutment of the dam connecting to the ring grouting around the tunnel plug need not extend more than 100 m into the river bank beyond the diversion tunnels.

The effect of the drainage tunnel on the seepage discharge is discussed in Section E-7.

E-5 RIGHT RIVER BANK

Analysis of seepage around the right abutment of the dam is complicated by the presence of the deep spillway excavation, with a gravel drain beneath the Spillway chute, both of which act as drains depressing the phreatic surface locally. The three dimensional problem was solved by analyzing the seepage flow parallel to the spillway chute through three sections located as shown on Plate E-1. For each of these sections, the shape of the phreatic surface was determined and was superimposed on two sections on the left side of and perpendicular to the Spillway chute. The analysis was continued in the same manner described for the left river bank. This method imposes flow either perpendicular to or parallel to the Spillway and as such it produces a conservatively high phreatic surface. The actual seepage flow will follow a complex, transverse but direct path towards the Spillway excavation from the Spillway approach channel with a corresponding lower phreatic surface.

The results of the analysis are shown on Plates E-7 to E-11. The first three plates show the flow parallel to the Spillway chute and the last two plates show the flow perpendicular to the Spillway chute.

The analysis showed that the phreatic surface for flow through the river bank between the dam abutment and the Spillway will be low. The effect of the grout curtain was not analyzed because seepage in this region will be low. No analysis of the seepage around the right side of the Spillway was carried out since the water head is less on the

right side of the Spillway than it is on the left side of the Spillway and will produce a lower phreatic surface and corresponding low seepage flow. Based on the results of the left river bank the grout curtain will be extended 200 m beyond the right abutment of the Spillway.

E-6 DAM

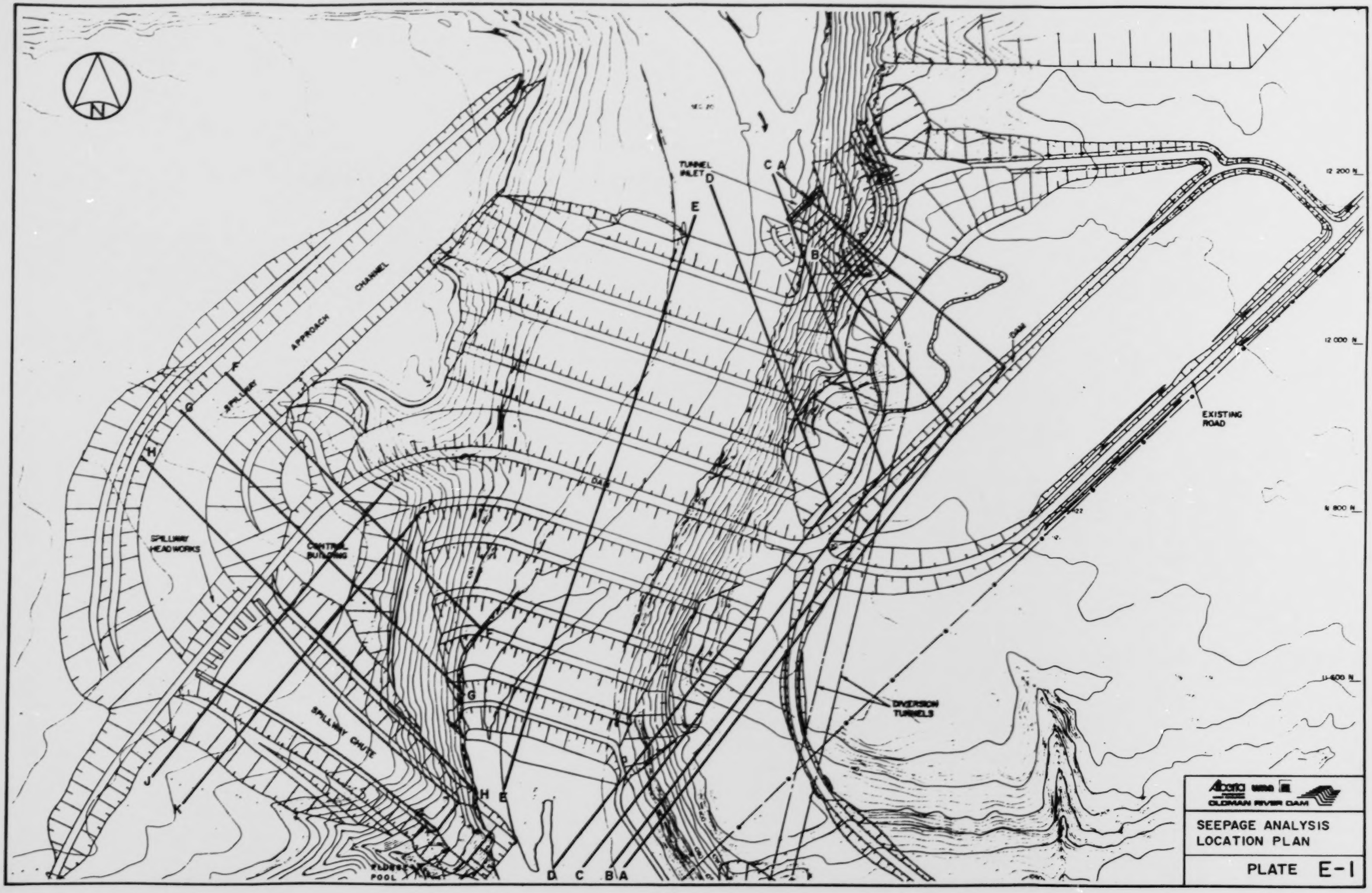
The width of the filter and drain zones immediately downstream of the core was varied to determine the minimum width that will result in acceptable hydraulic gradients in the rock foundation that will not be consolidated by shallow grouting. The analysis is very sensitive to the permeability values assigned to the rock foundation in the core trench. The actual permeability should be low as a result of grouting. It was determined however that the exit gradient will be less than 0.5 for the 9 m wide filter and drain zone contact proposed. The results are shown on Plates E-12 to E-13.

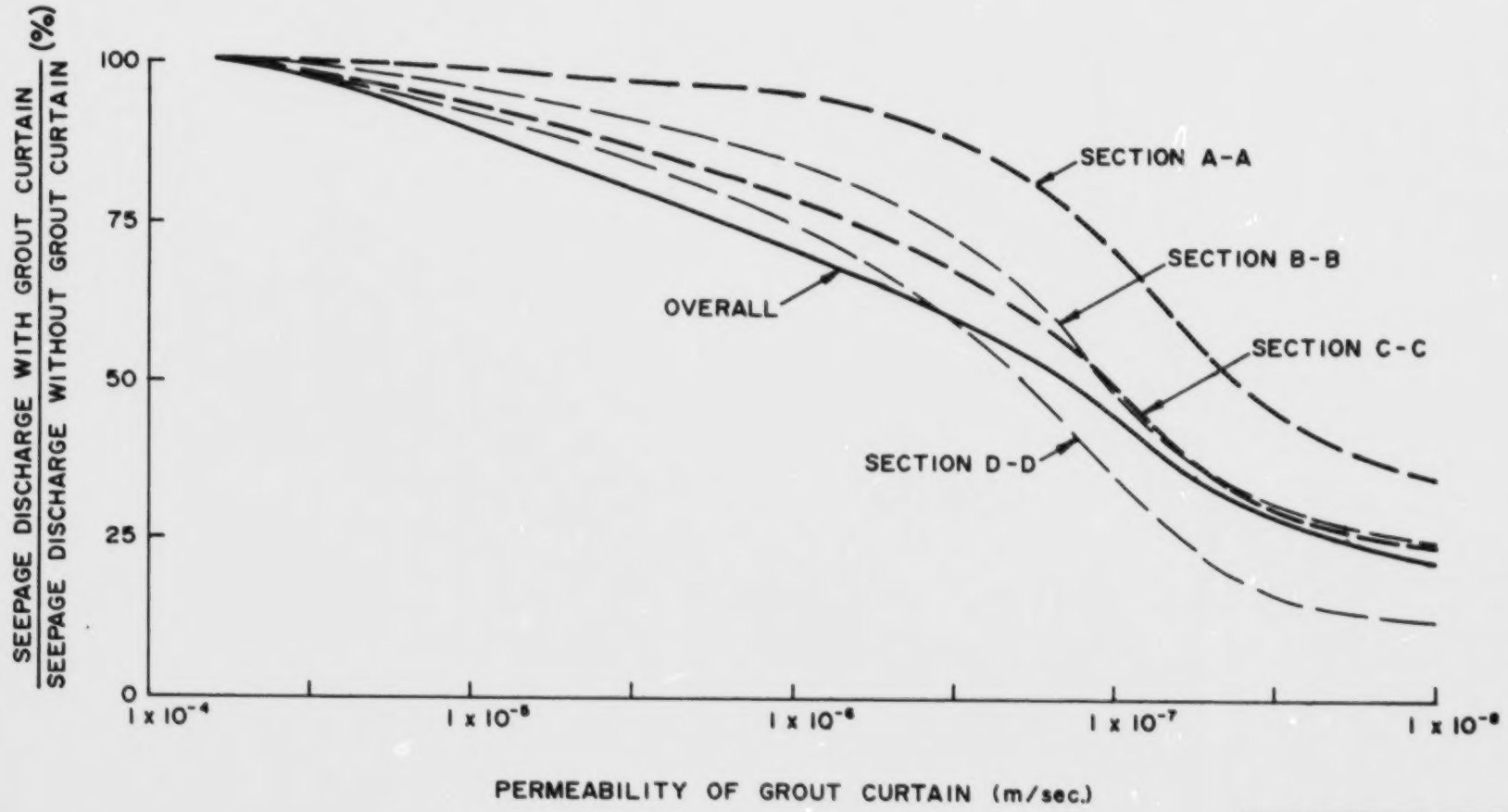
E-7 LEFT DRAINAGE TUNNEL

A seepage analysis was performed to determine the most effective location of the left bank drainage tunnels with respect to the grout curtain, the dam and the diversion tunnels.

Six different cases were analyzed using varying depths of grout curtain and varying locations of the drainage tunnel with respect to the grout curtain.

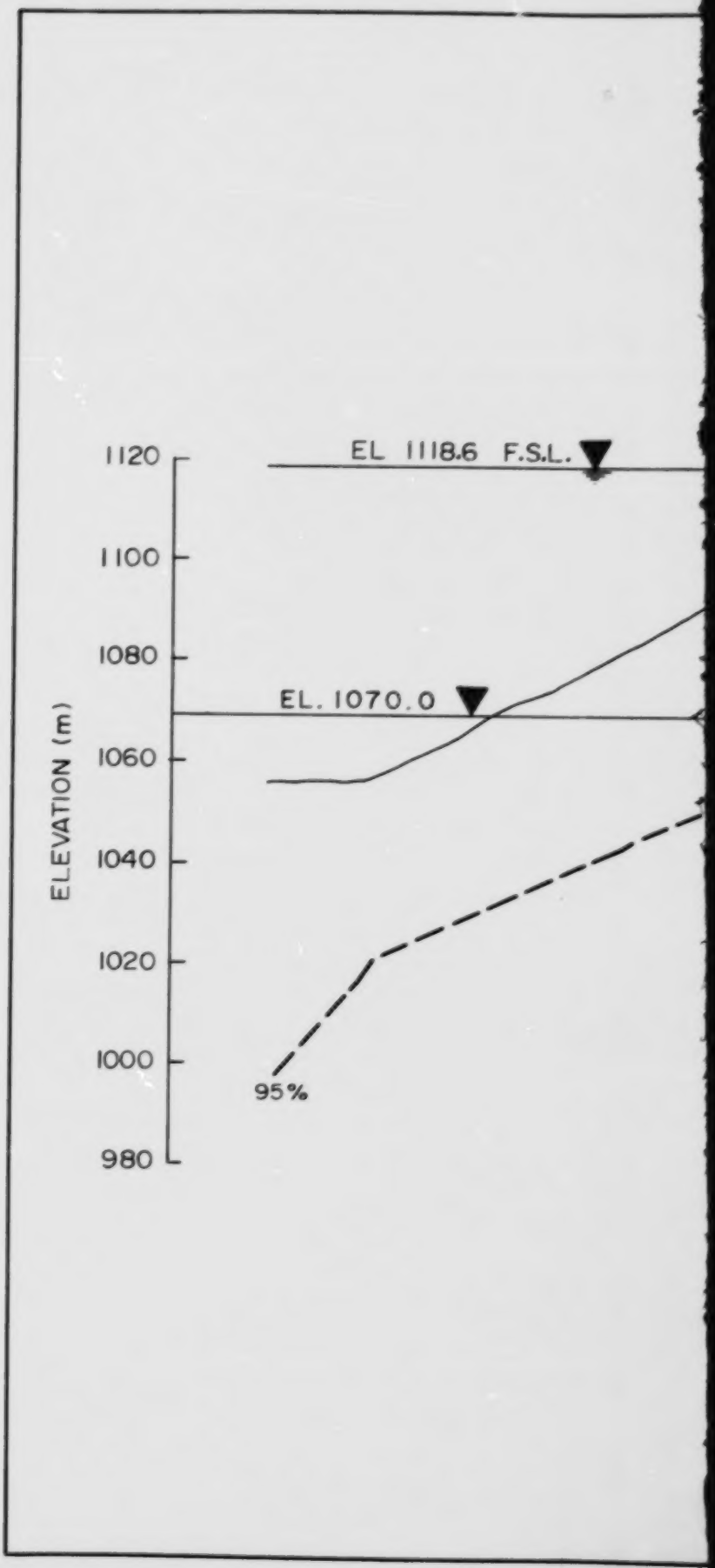
The results of the analyses are shown in Plates E-14 to E-19. The equipotential lines indicate that within the range of drainage tunnel locations and the depths of grout curtain considered the effect on the hydraulic gradients is minimal. The left drainage tunnel was therefore, located by considering that there is little opportunity to move the drainage tunnel away from the grout curtain because of its proximity to the diversion tunnels.

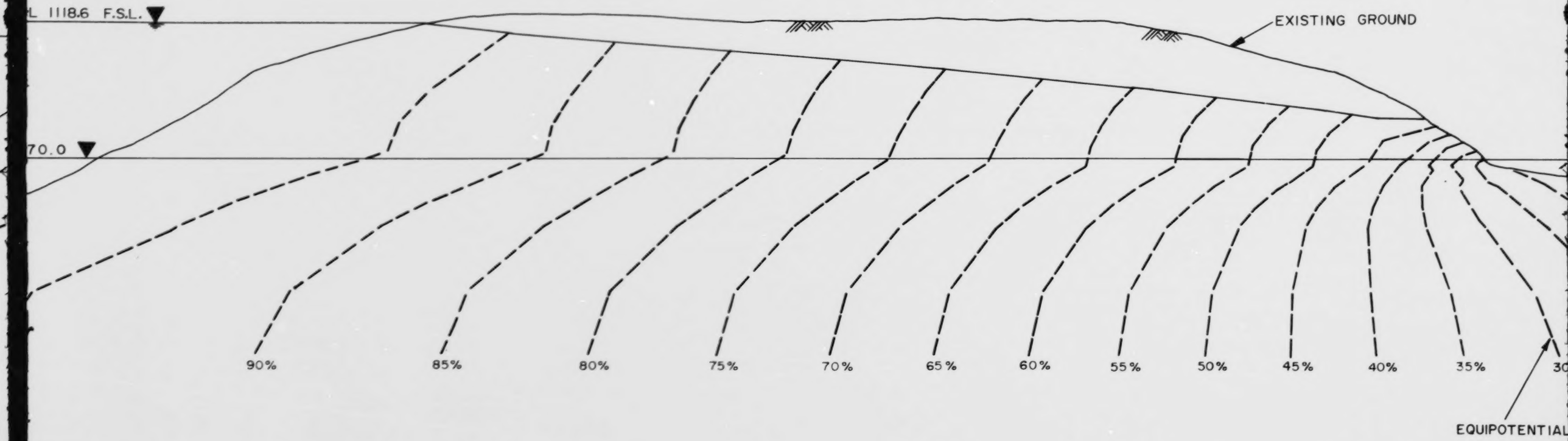

N
TUNNEL INLET
A
B
C
D
E
F
G
H
J
K
APPROACH CHANNEL
SPILLWAY
SPILLWAY HEADWORKS
CONTROL BUILDING
SPILLWAY CHUTE
FLIP BUCKET POOL
DAM
DIVERSION TUNNELS
EXISTING ROAD
12 200 N
12 000 N
11 800 N
11 600 N
Alberta Power
OLDMAN RIVER DAM
SEEPAGE ANALYSIS
LOCATION PLAN
PLATE E-1



REDUCTION OF SEEPAGE DISCHARGE WITH GROUT CURTAIN.

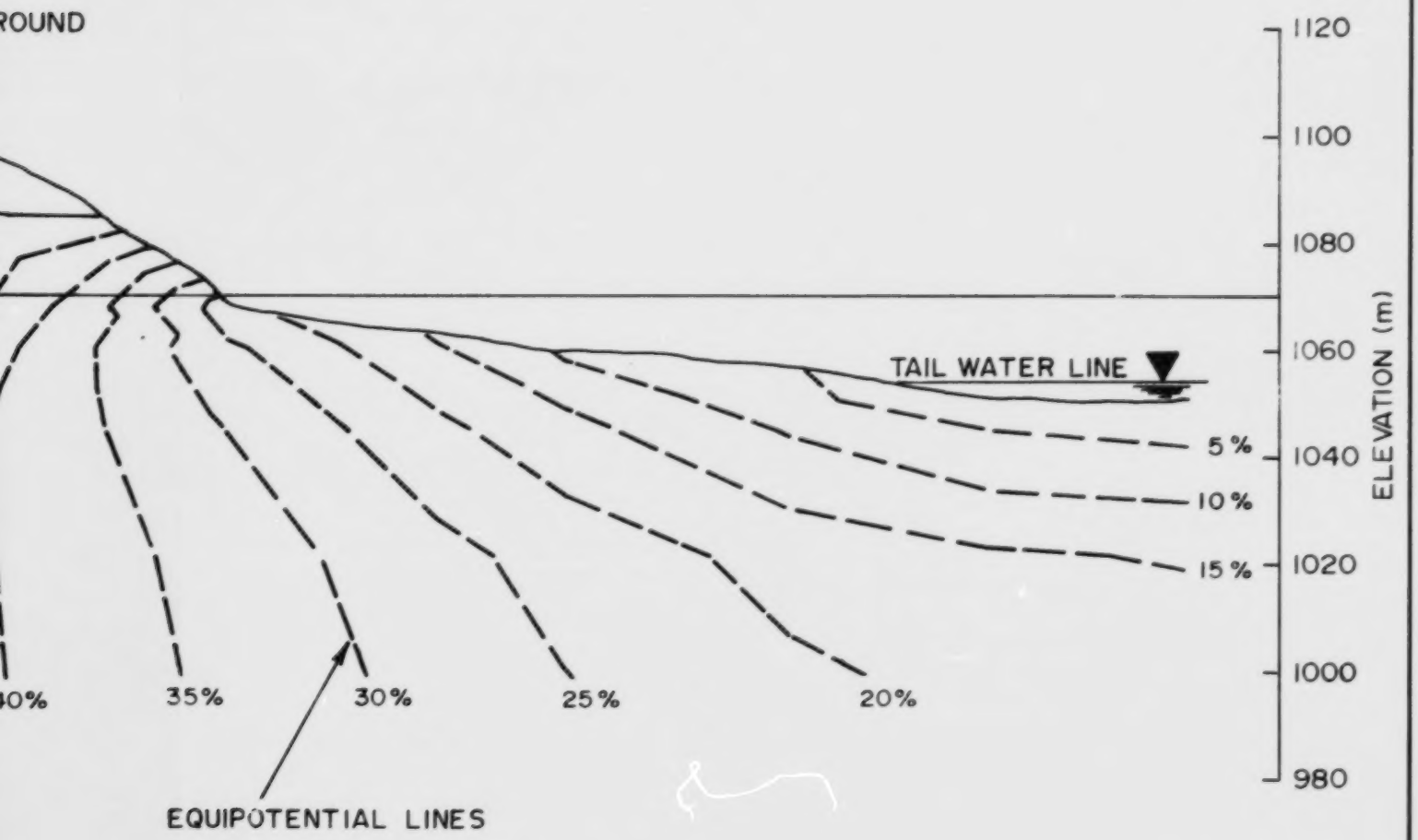
PLATE E-2


EL 1118.6 F.S.L.
EL. 1070.0
95%
ELEVATION (m)
1120
1100
1080
1060
1040
1020
1000
980


L 1118.6 F.S.L.
70.0
EXISTING GROUND
90%
85%
80%
75%
70%
65%
60%
55%
50%
45%
40%
35%
EQUIPOTENTIAL

MATERIAL PERMEABILITIES (m/sec)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06

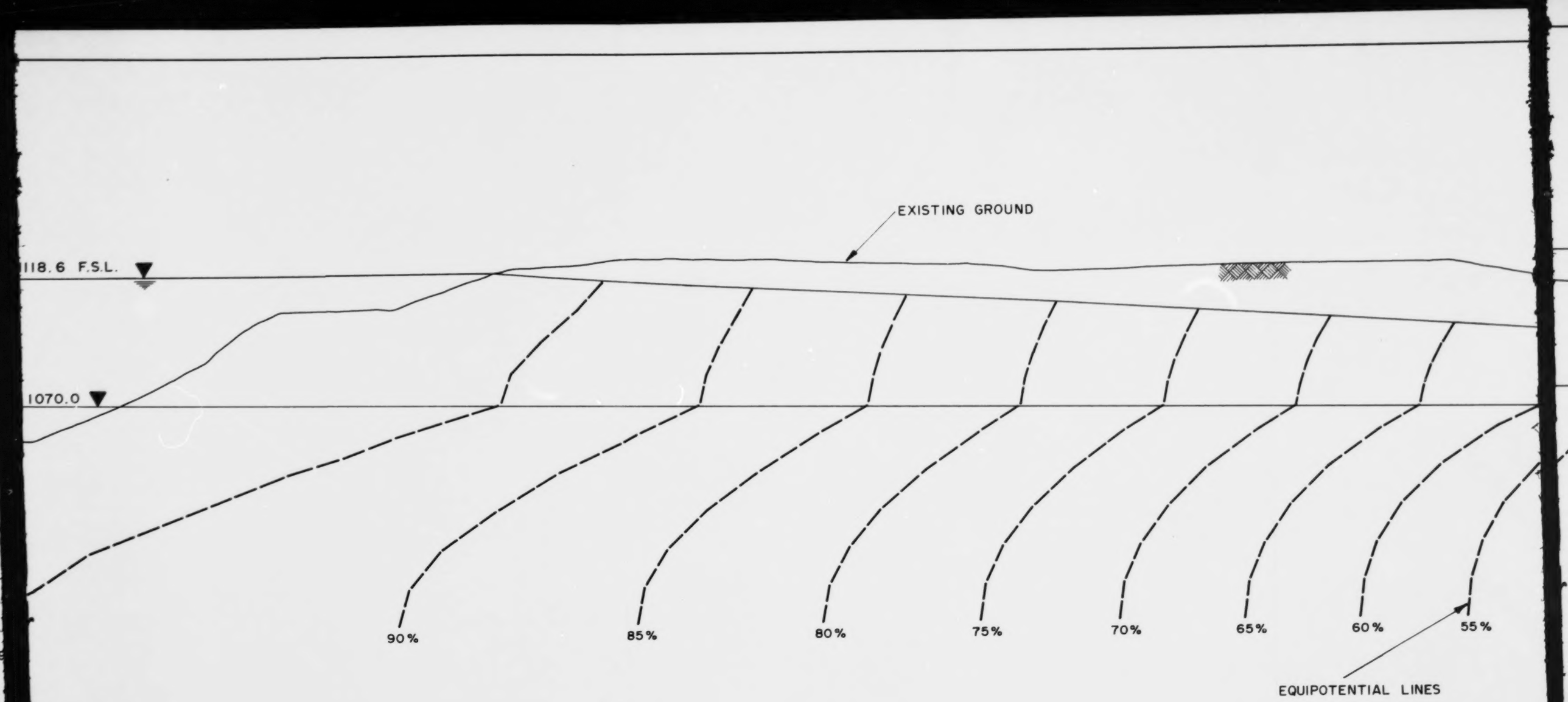


OLDMAN RIVER DAM

LEFT RIVER BANK
SECTION A

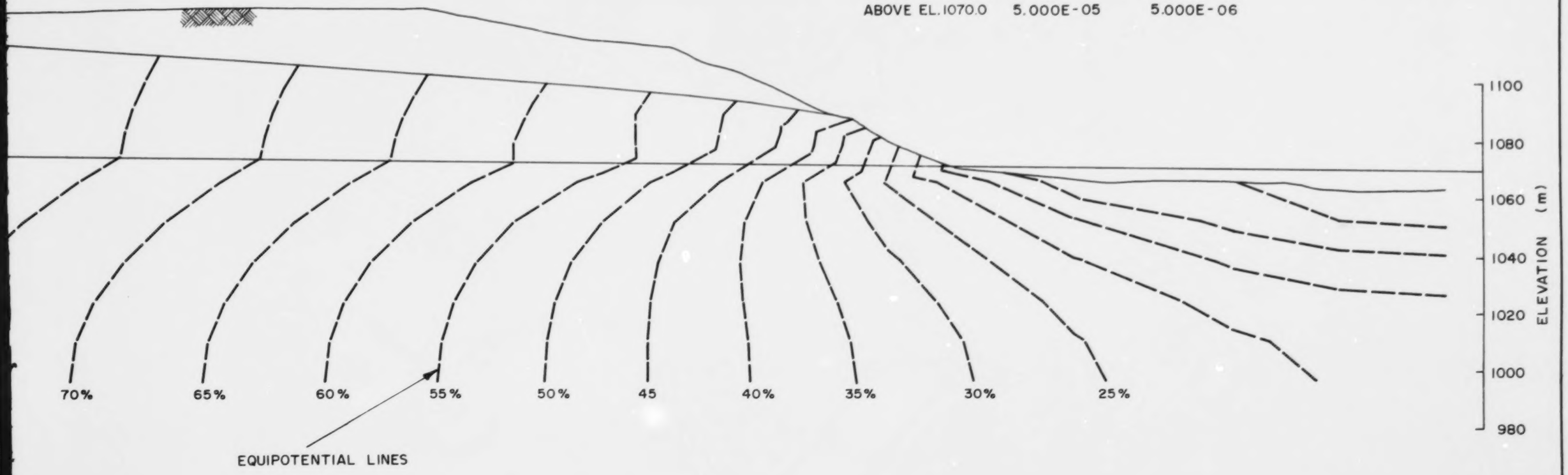
PLATE E-3

1100
1080
1060
1040
1020
1000
980
ELEVATION (m)


EXISTING GROUND
118.6 F.S.L.
1070.0
90%
85%
80%
75%
70%
65%
60%
55%
EQUIPOTENTIAL LINES

MATERIAL PERMEABILITIES (m/sec.)

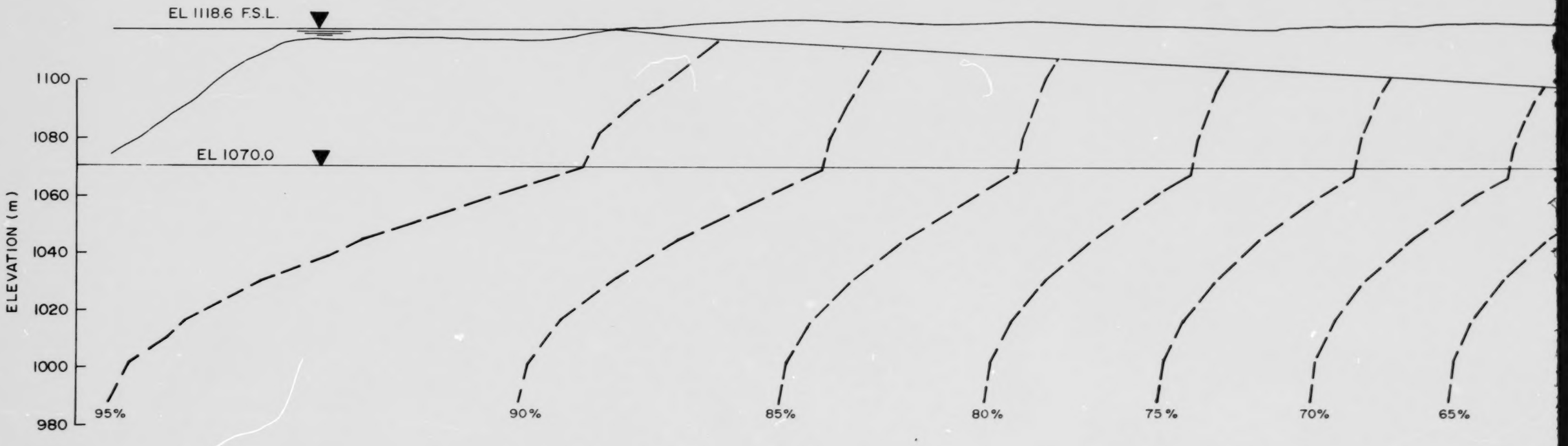
	K1	K2
BELOW EL.1070.0	5.000E-07	5.000E-08
ABOVE EL.1070.0	5.000E-05	5.000E-06



ADAMS ARMS
OLDMAN RIVER DAM

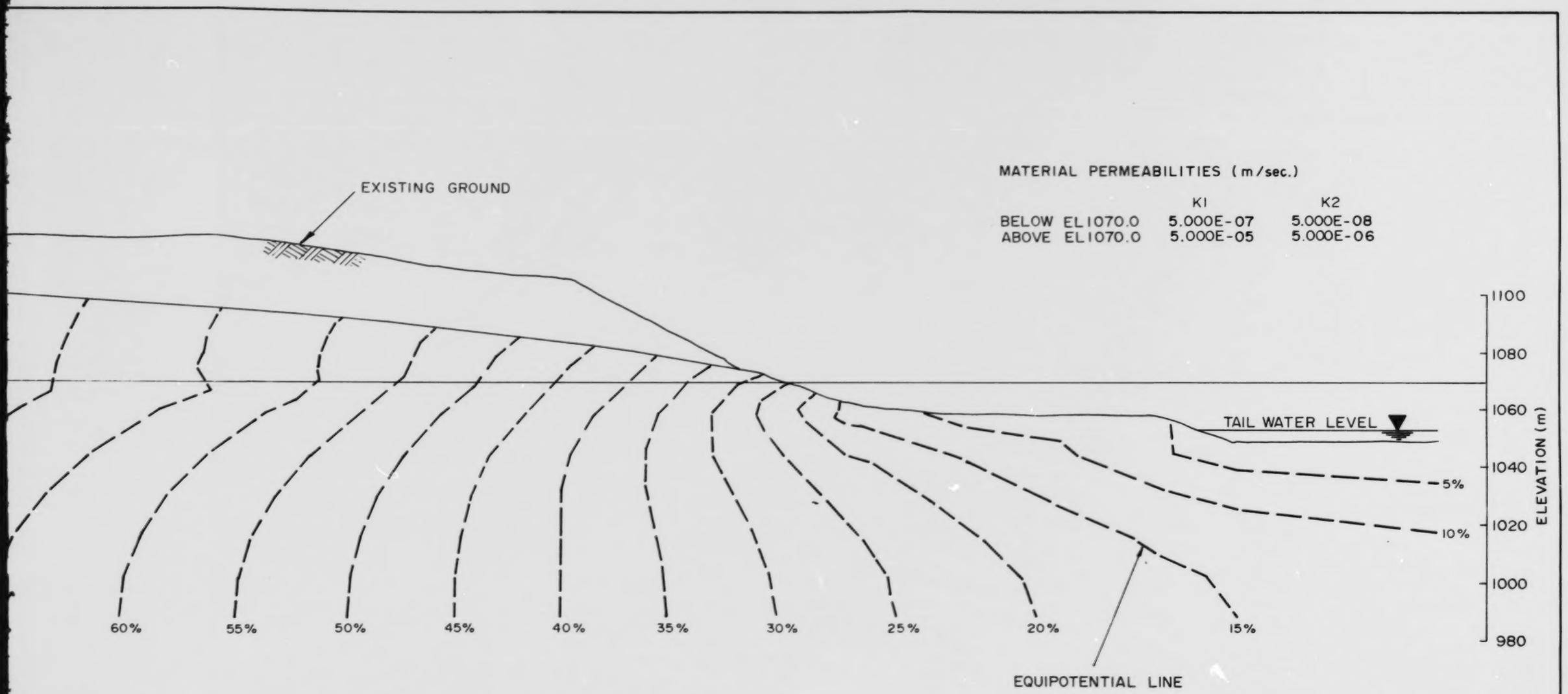
LEFT RIVER BANK
SECTION B

PLATE E-4


EL 1118.6 F.S.L.
EL 1070.0
ELEVATION (m)
1100
1080
1060
1040
1020
1000
980
95%
90%
85%
80%
75%
70%
65%

MATERIAL PERMEABILITIES (m/sec.)

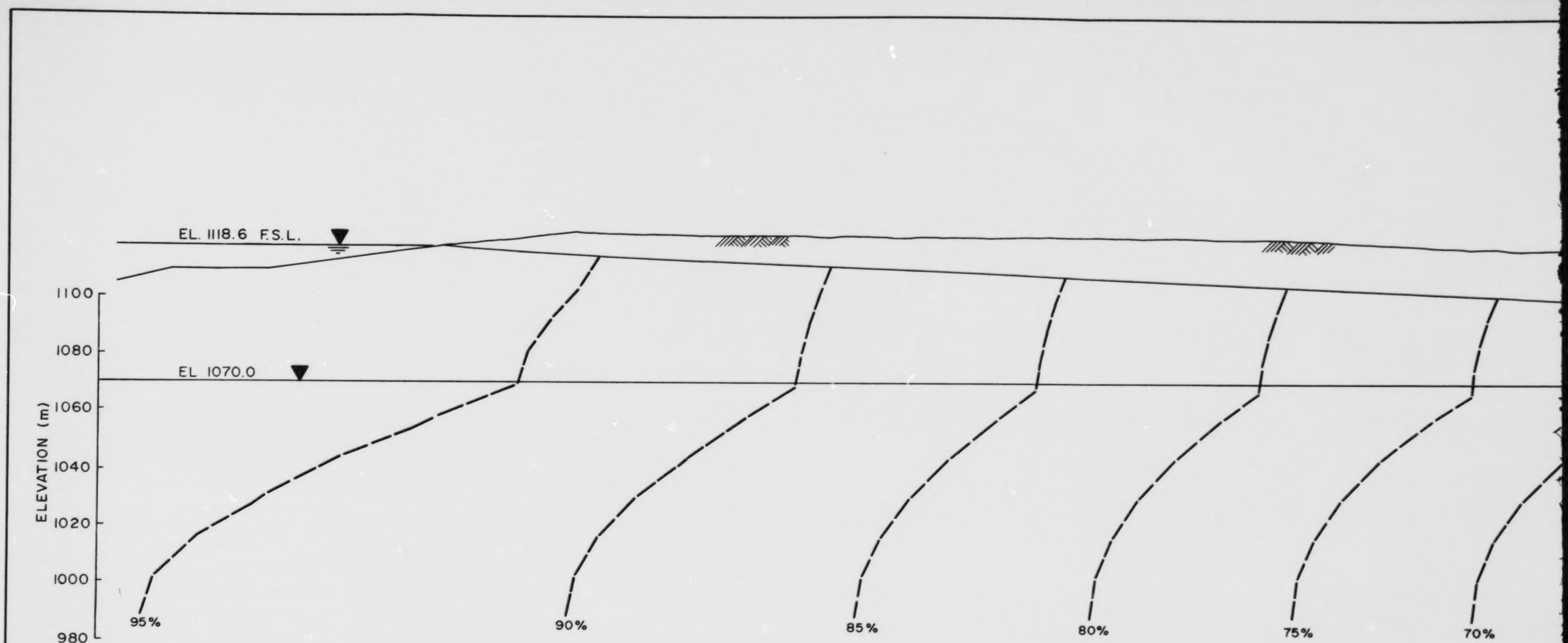
	K1	K2
BELOW EL1070.0	5.000E-07	5.000E-08
ABOVE EL1070.0	5.000E-05	5.000E-06



Alberta Power
OLDMAN RIVER DAM

LEFT RIVER BANK
SECTION C

PLATE E-5


EL. 1118.6 F.S.L.
EL 1070.0
1100
1080
1060
1040
1020
1000
980
ELEVATION (m)
95%
90%
85%
80%
75%
70%

MATERIAL PERMEABILITIES (m/sec)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E
ABOVE EL. 1070.0	5.000E-05	5.000E

EXISTING GROUND

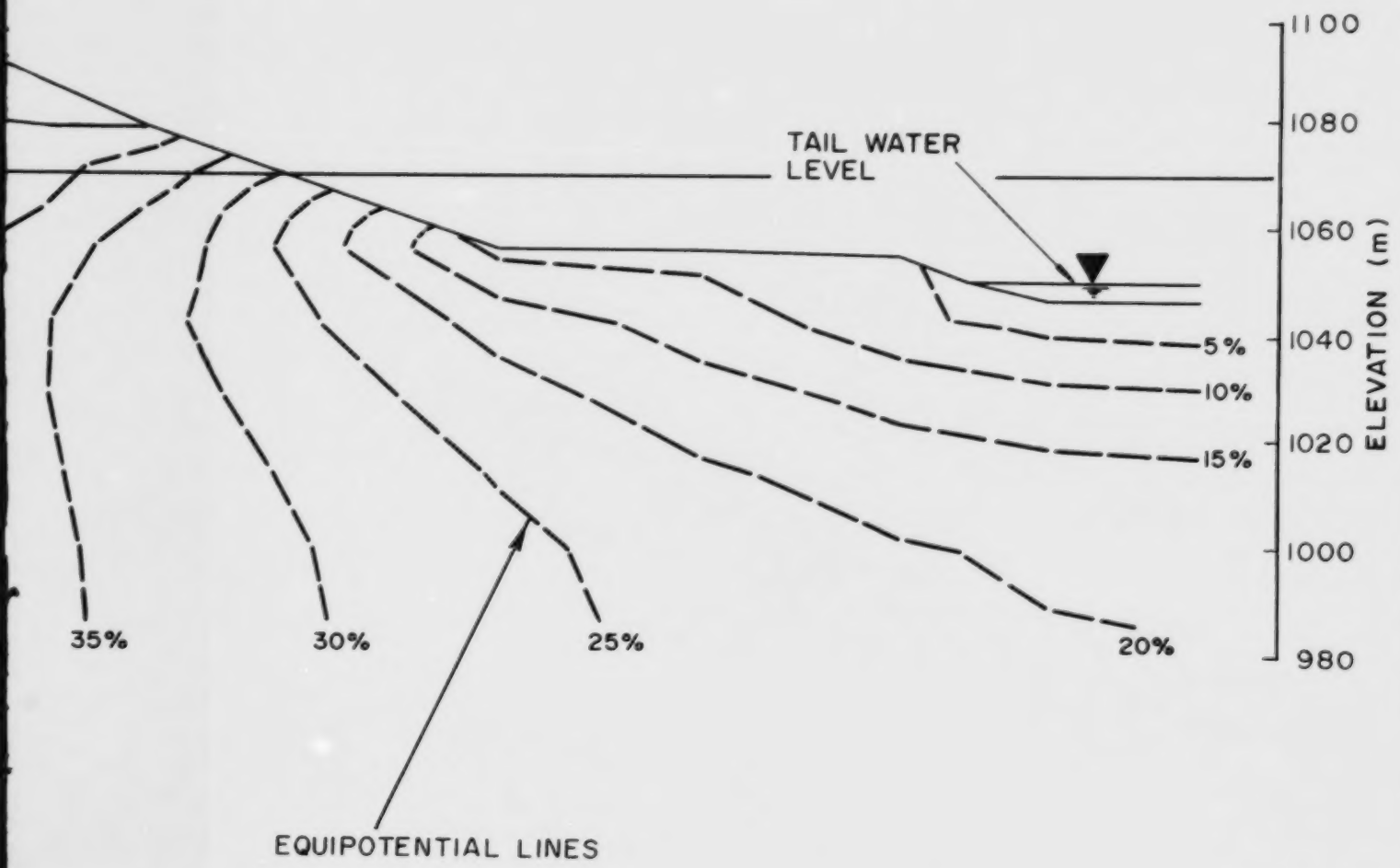
TAIL W
LEVEL

75% 70% 65% 60% 55% 50% 45% 40% 35% 30% 25%

EQUIPOTENTIAL LINES

MATERIAL PERMEABILITIES (m/sec)

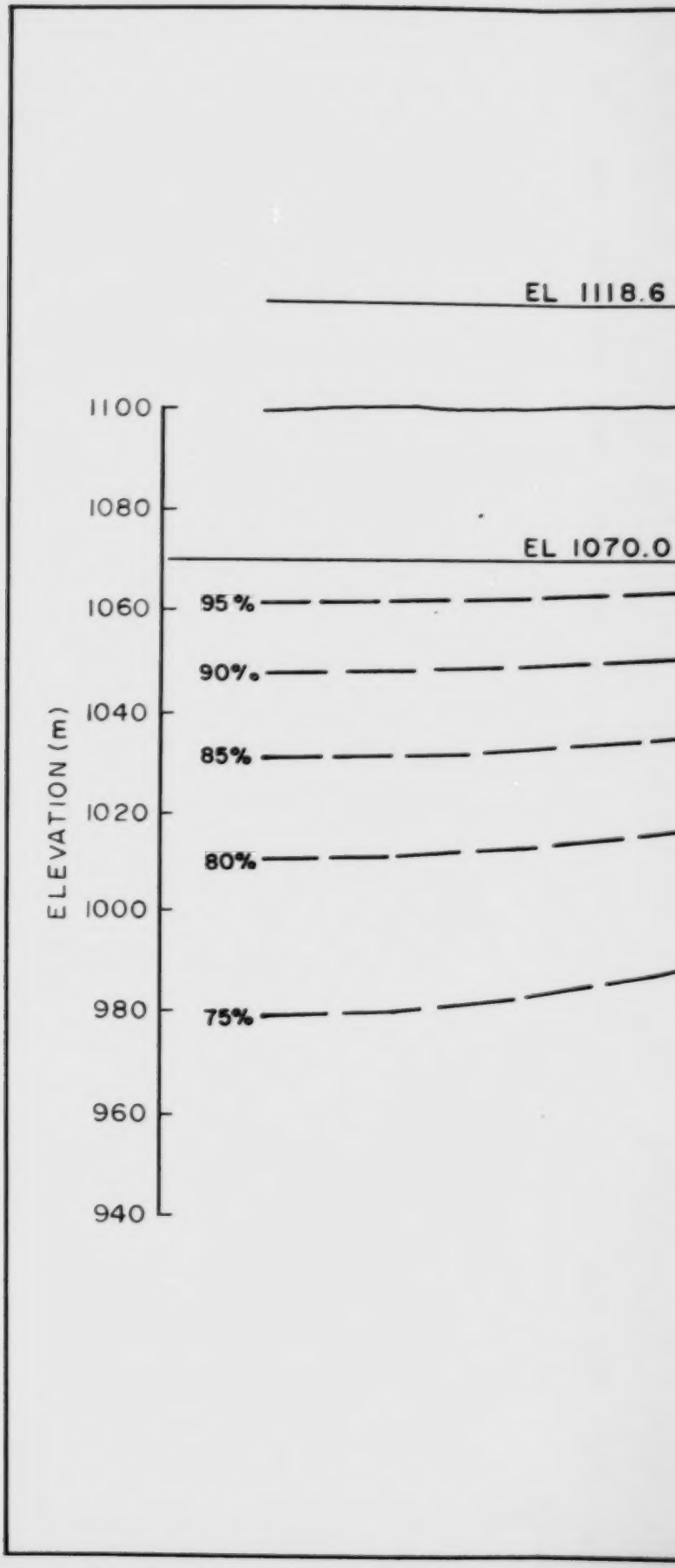
	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06



OLDMAN RIVER DAM

LEFT RIVER BANK
SECTION D

PLATE E-6


EL 1118.6
EL 1070.0
95%
90%
85%
80%
75%
ELEVATION (m)
1100
1080
1060
1040
1020
1000
980
960
940

MATERIAL P

BELOW EL 10

ABOVE EL 10

EXISTING GROUND

8.6 F.S.L

0.0

70%

65%

60%

55%

50%

45%

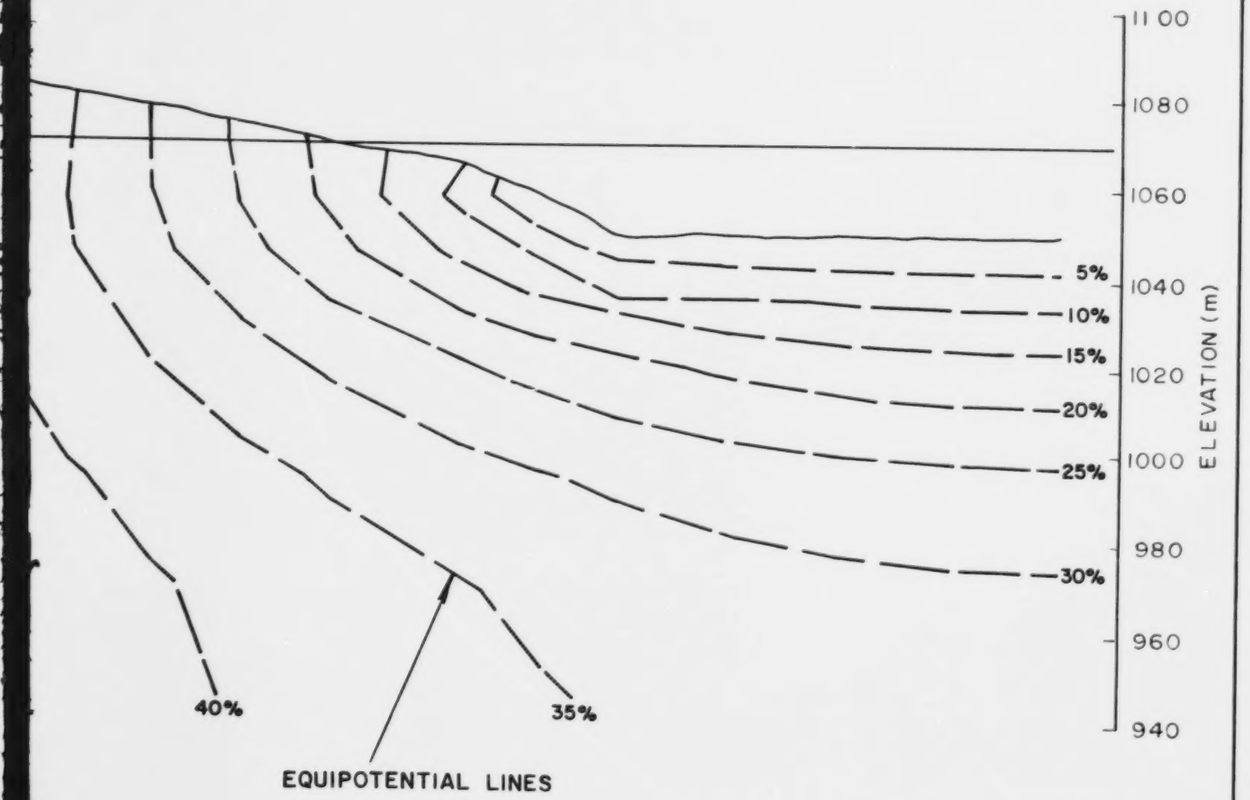
40%

35%

EQUIPOTENTIAL LINES

MATERIAL PERMEABILITIES (m/sec.)

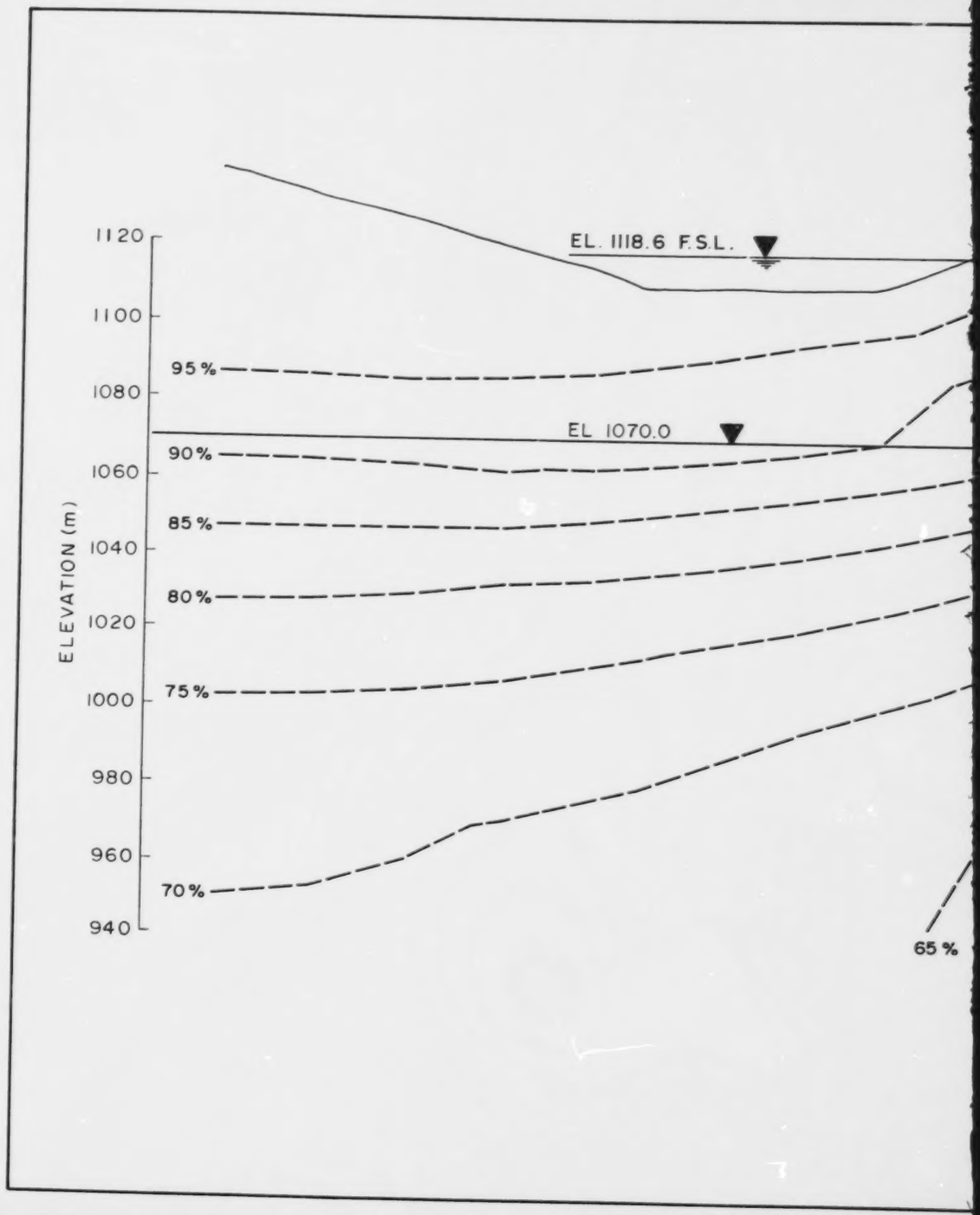
	K1	K2
BELOW EL 1070.0	1.000E-07	1.000E-08
ABOVE EL 1070.0	1.000E-06	1.000E-07



Alberta Power
OLDMAN RIVER DAM

RIGHT RIVER BANK
SECTION F

PLATE E-7



MATERIAL PERMEABILITIES (

	K I
BELOW EL 1070.0	1.000E-0
ABOVE EL 1070.0	1.000E-0

1118.6 F.S.L.

1070.0

EXISTING GROUND

65%

60%

55%

50%

45%

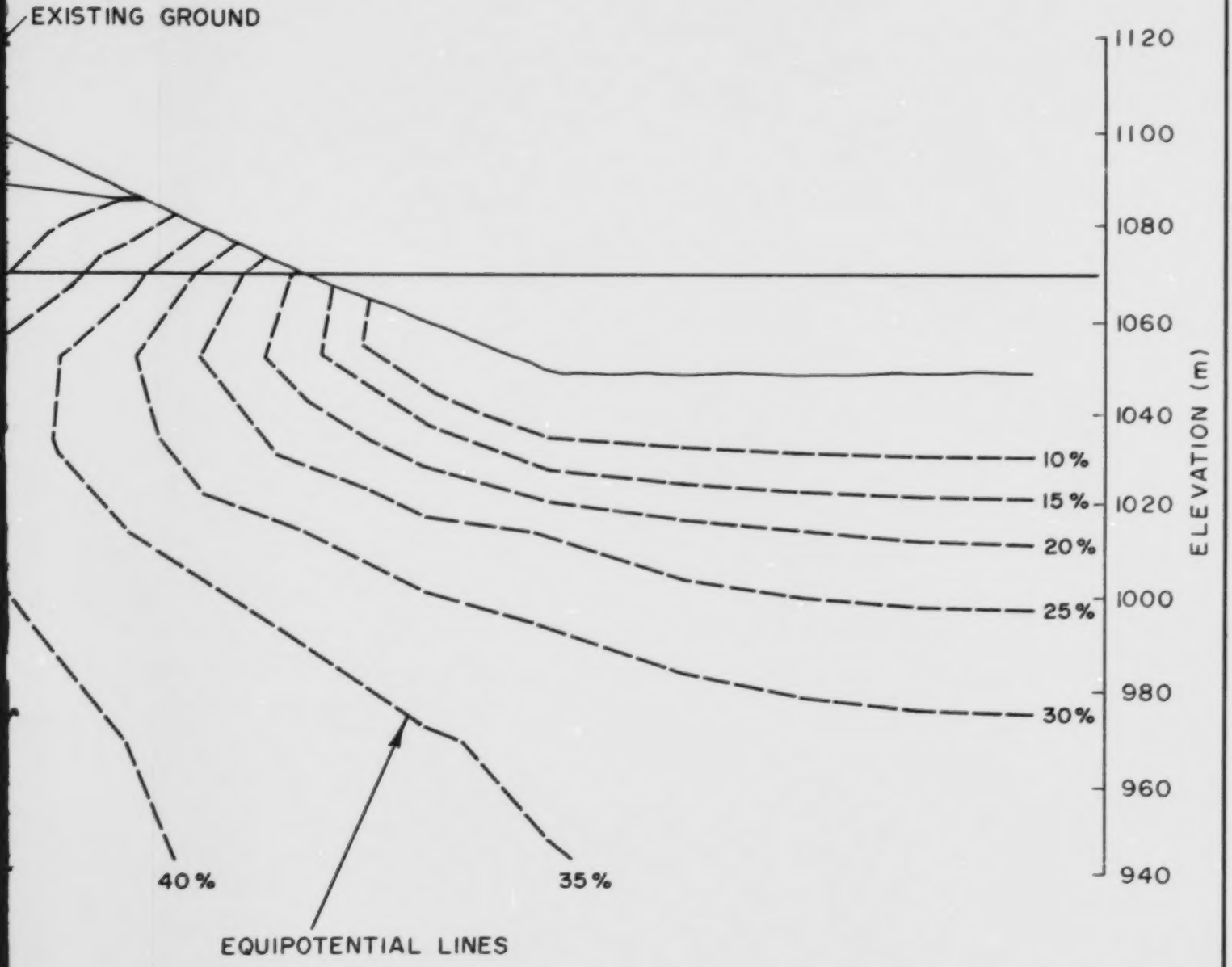
40%

35%

EQUIPOTENTIAL LINES

MATERIAL PERMEABILITIES (m/sec.)

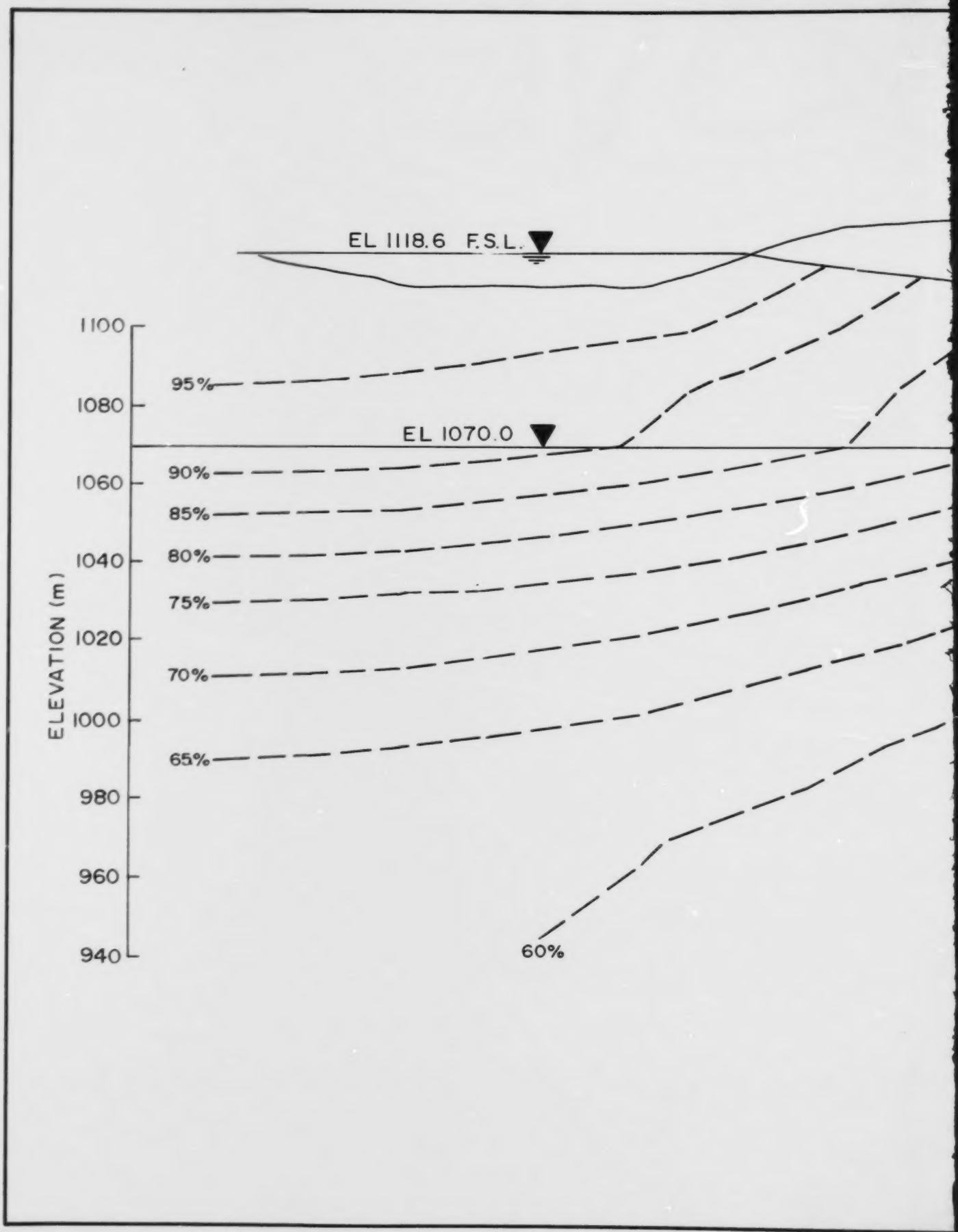
	K1	K2
BELOW EL 1070.0	1.000E-07	1.000E-08
ABOVE EL 1070.0	1.000E-06	1.000E-07



Alberta Utility Consultants
OLDMAN RIVER DAM

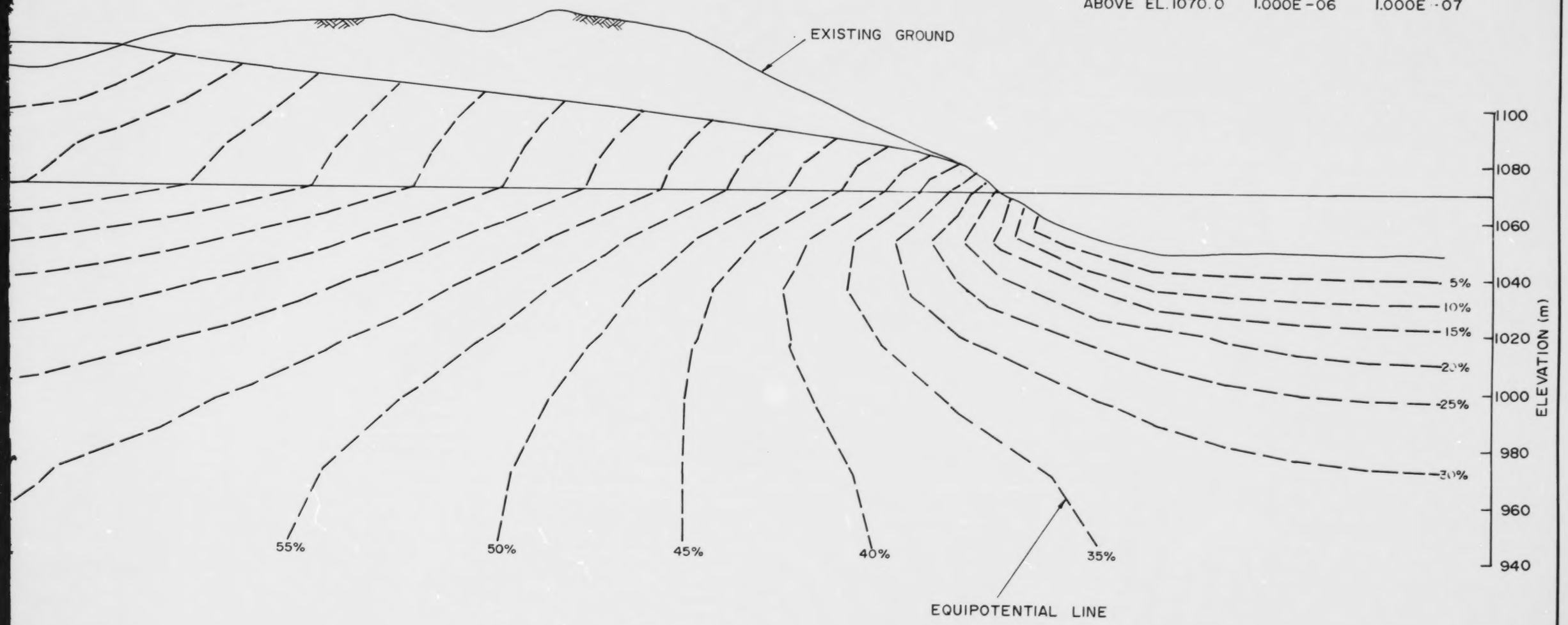
RIGHT RIVER BANK
SECTION G

PLATE E-8


EL 1118.6 F.S.L.
EL 1070.0
95%
90%
85%
80%
75%
70%
65%
60%
1100
1080
1060
1040
1020
1000
980
960
940
ELEVATION (m)

MATERIAL PERMEABILITIES (m/sec.)

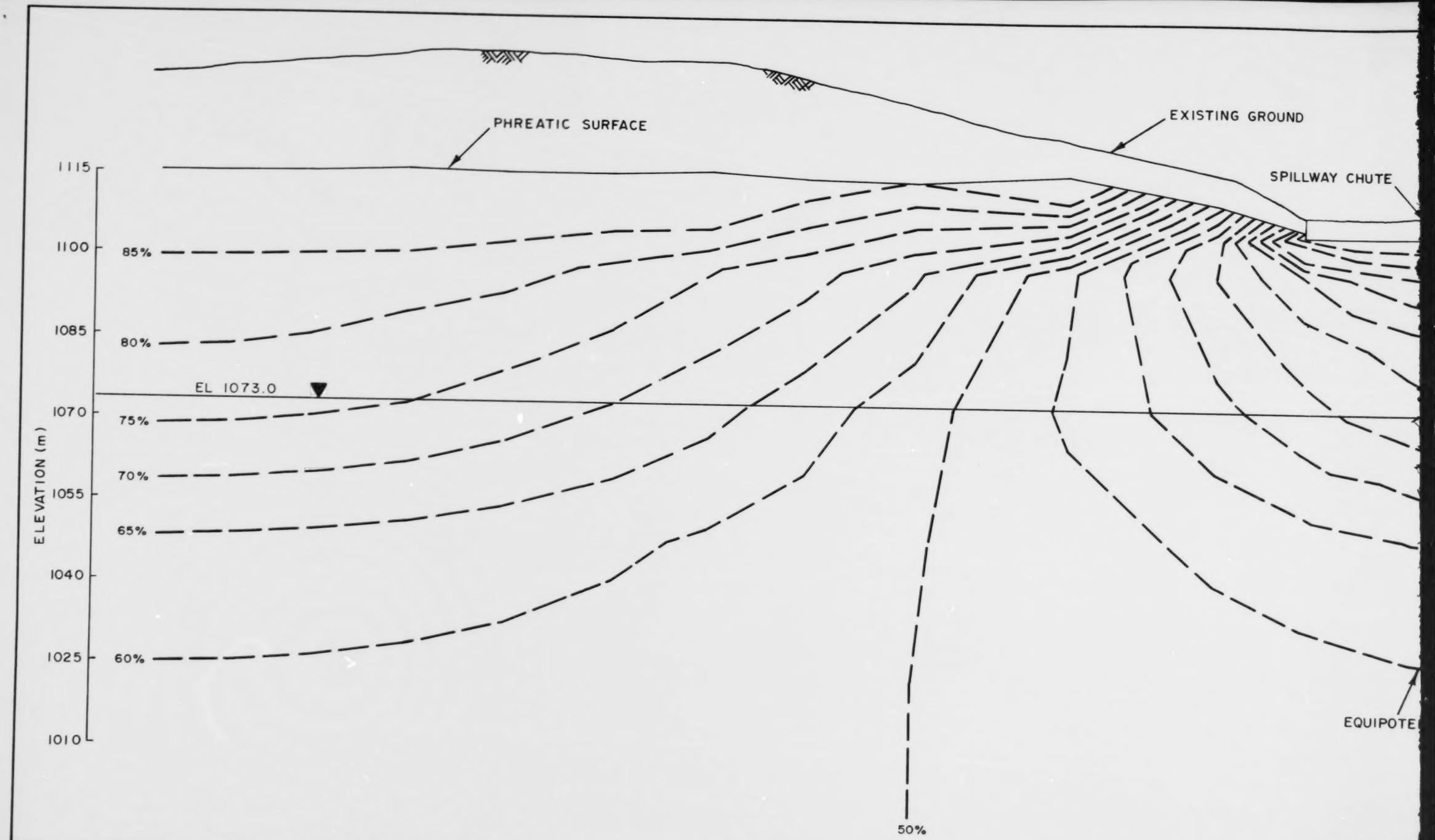
	K1	K2
BELOW EL. 1070.0	1.000E-07	1.000E-08
ABOVE EL. 1070.0	1.000E-06	1.000E-07



OLDMAN RIVER DAM

RIGHT RIVER BANK
SECTION H

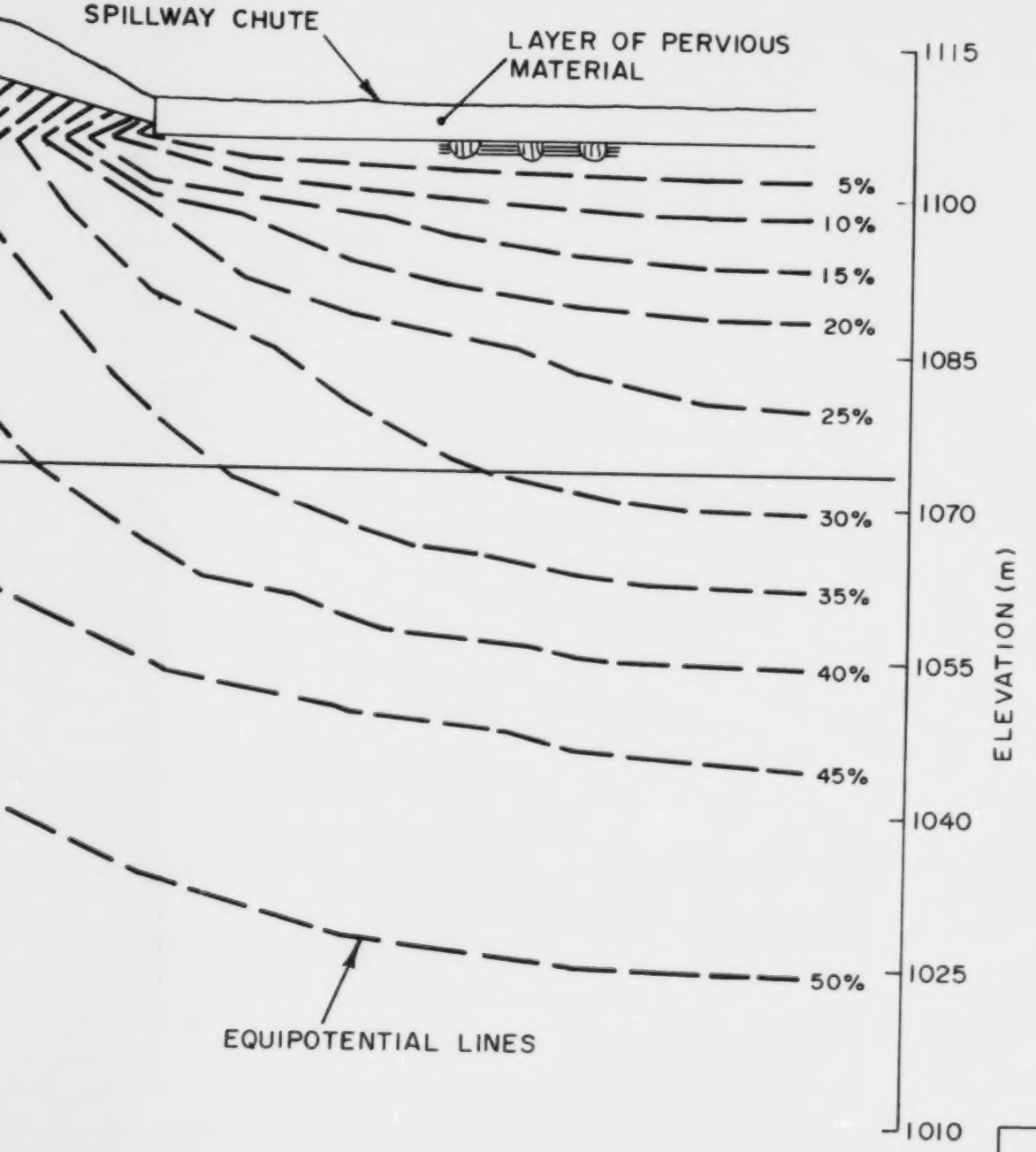
PLATE E-9


PHREATIC SURFACE
EXISTING GROUND
SPILLWAY CHUTE
EL 1073.0
85%
80%
75%
70%
65%
60%
50%
1115
1100
1085
1070
1055
1040
1025
1010
ELEVATION (m)
EQUIPOTE

NG GROUND

MATERIAL PERMEABILITIES (m/sec)

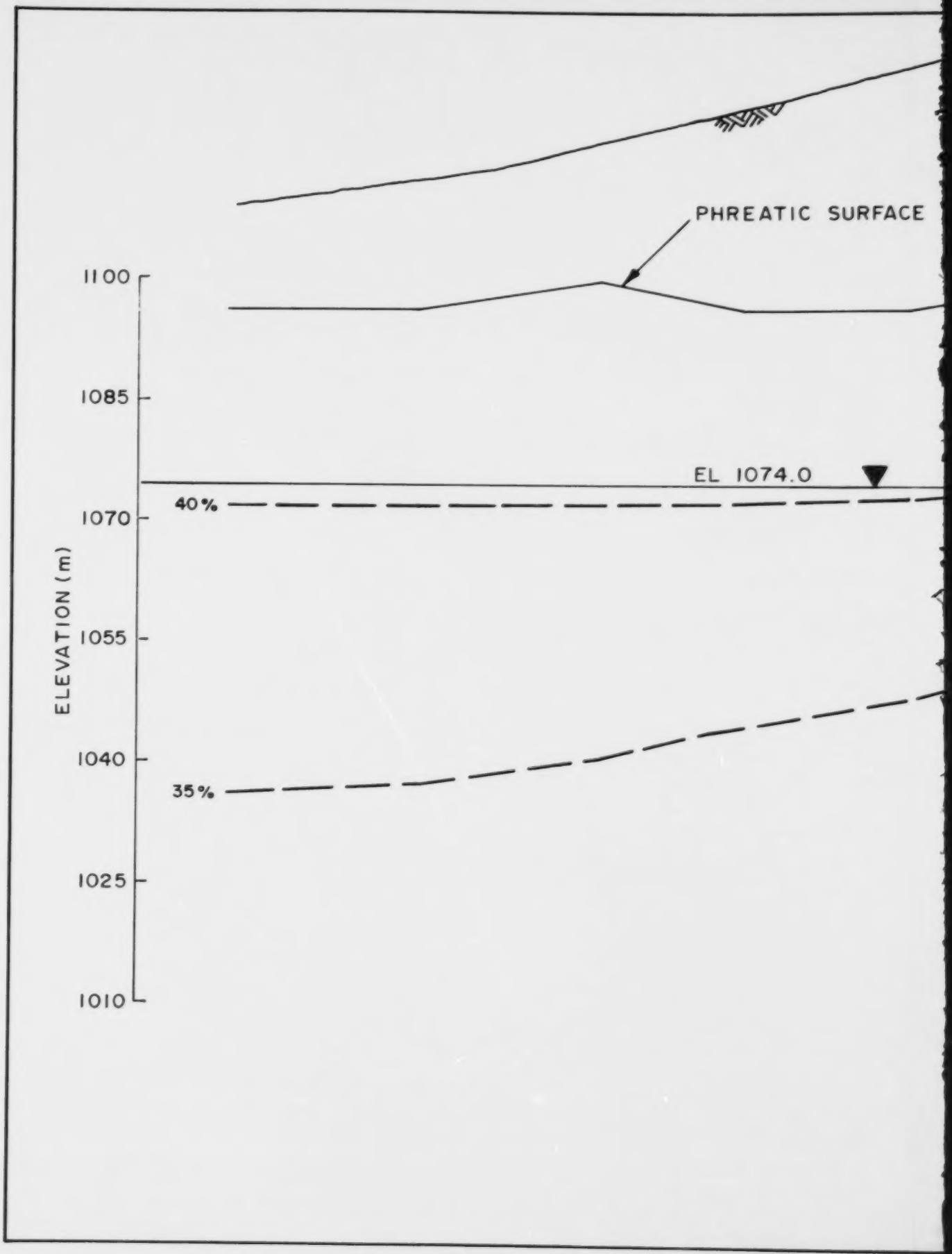
	K1	K2
BELOW EL. 1074.0	1.000E-07	1.000E-08
ABOVE EL. 1074.0	1.000E-06	1.000E-07



Alcoa Umo
OLOMAN RIVER DAM

RIGHT RIVER BANK
SECTION J

PLATE E-10


PHREATIC SURFACE
1100
1085
EL 1074.0
40%
1070
ELEVATION (m)
1055
1040
35%
1025
1010

MATERIAL PERMEABILITIES (m/sec.)

	K1	K2
BELOW EL.1074.0	1.000E-07	1.000E-08
ABOVE EL.1074.0	1.000E-06	1.000E-07

EXISTING GROUND

PHREATIC SURFACE

SPILLWAY CHUTE

LAYER OF PERVIOUS MATERIAL

50%

5%

10%

15%

20%

25%

30%

EL 1074.0

1100
1085
1070
1055
1040
1025
1010

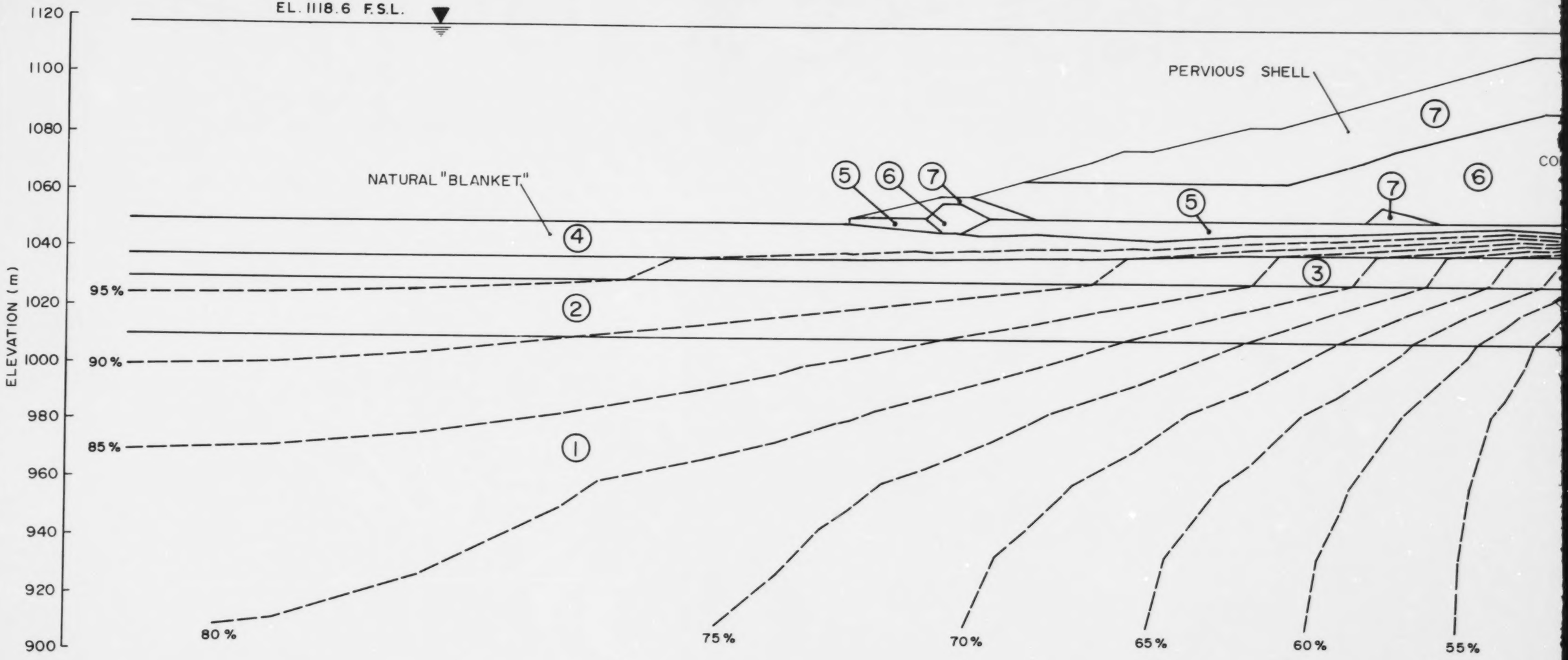
ELEVATION (m)

EQUIPOTENTIAL LINES

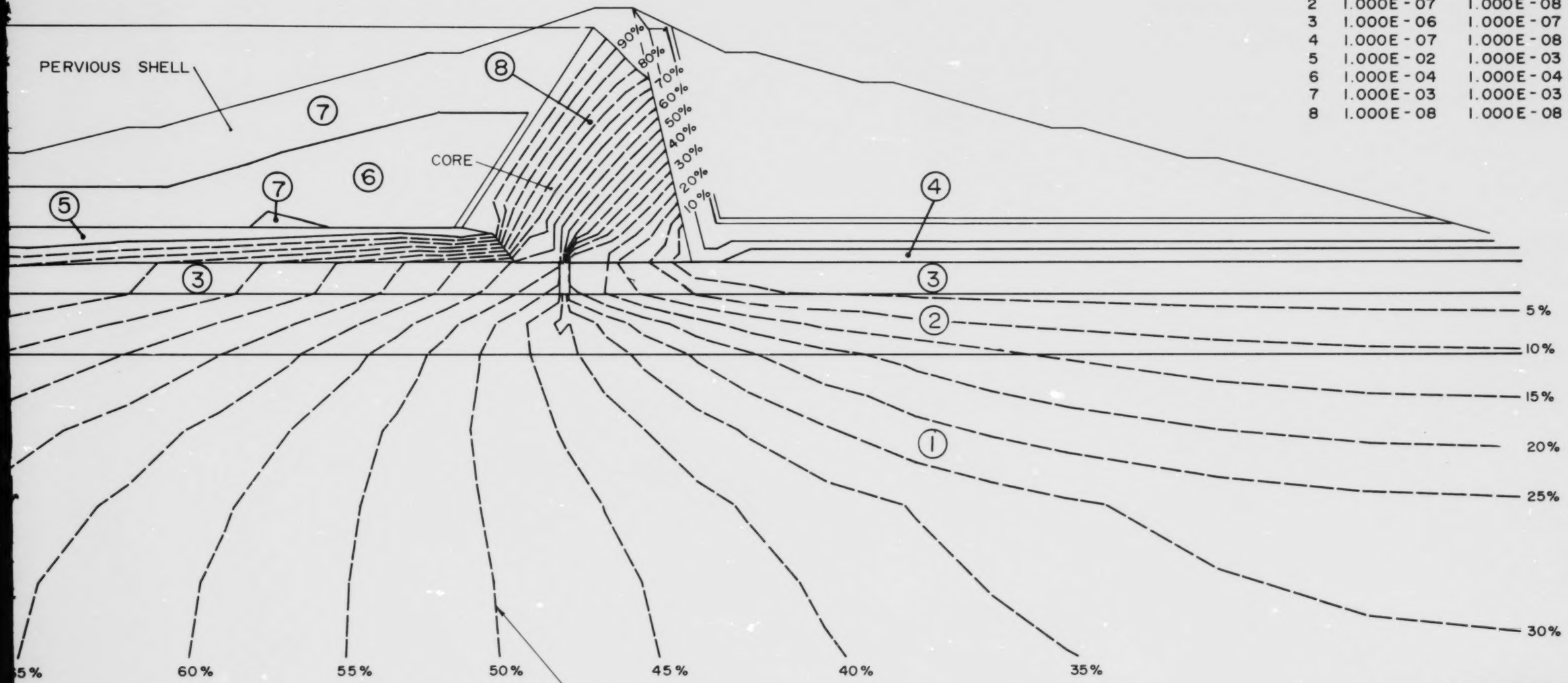
Alberta Environment
OLDMAN RIVER DAM

RIGHT RIVER BANK
SECTION K

PLATE E-11


EL. 1118.6 F.S.L.
PERVIOUS SHELL
NATURAL "BLANKET"
ELEVATION (m)
1120
1100
1080
1060
1040
1020
1000
980
960
940
920
900
95%
90%
85%
80%
75%
70%
65%
60%
55%
1
2
3
4
5
6
7

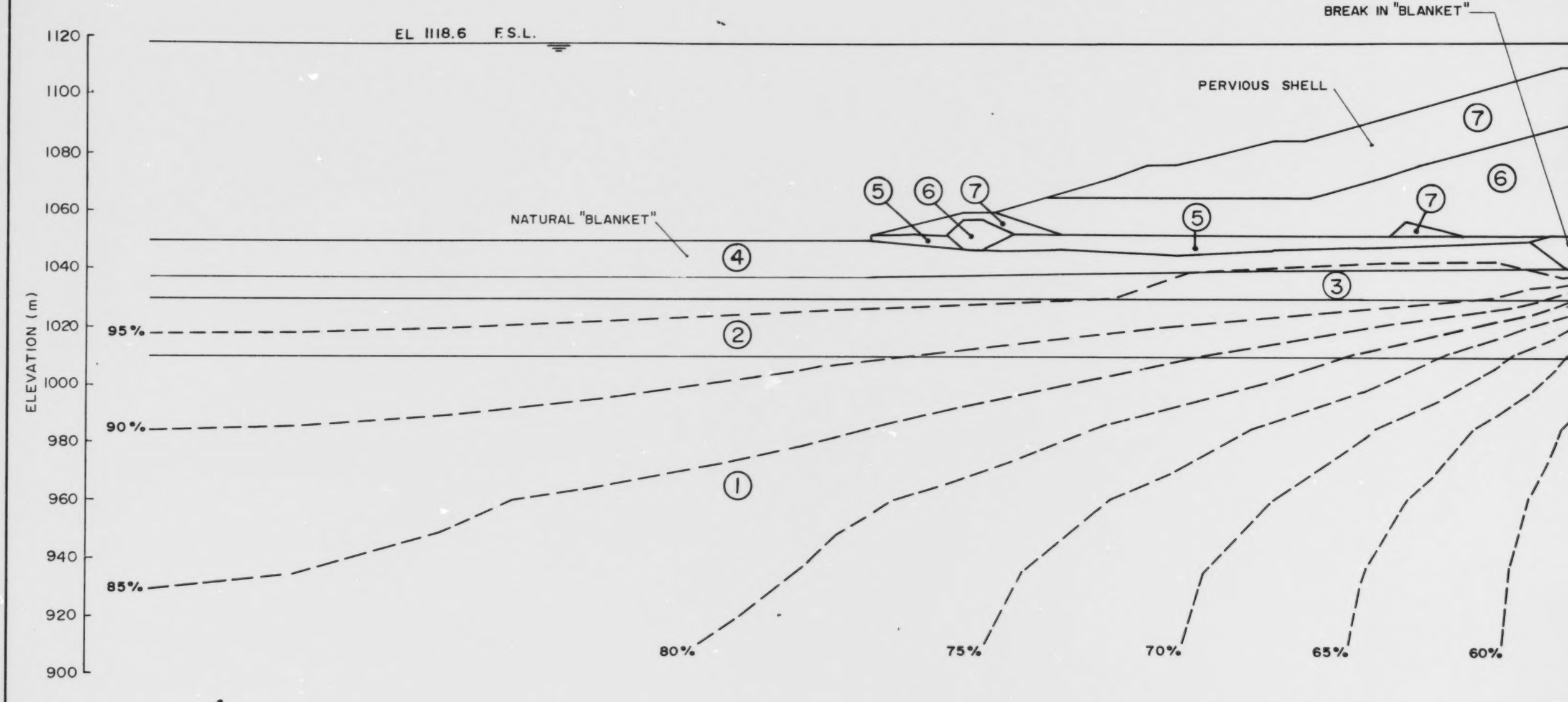
MATERIAL	PERMEABILITIES (m/sec.)	
	K1	K2
1	1.000E-07	1.000E-08
2	1.000E-07	1.000E-08
3	1.000E-06	1.000E-07
4	1.000E-07	1.000E-08
5	1.000E-02	1.000E-03
6	1.000E-04	1.000E-04
7	1.000E-03	1.000E-03
8	1.000E-08	1.000E-08



Alberta Power Limited
OLDMAN RIVER DAM

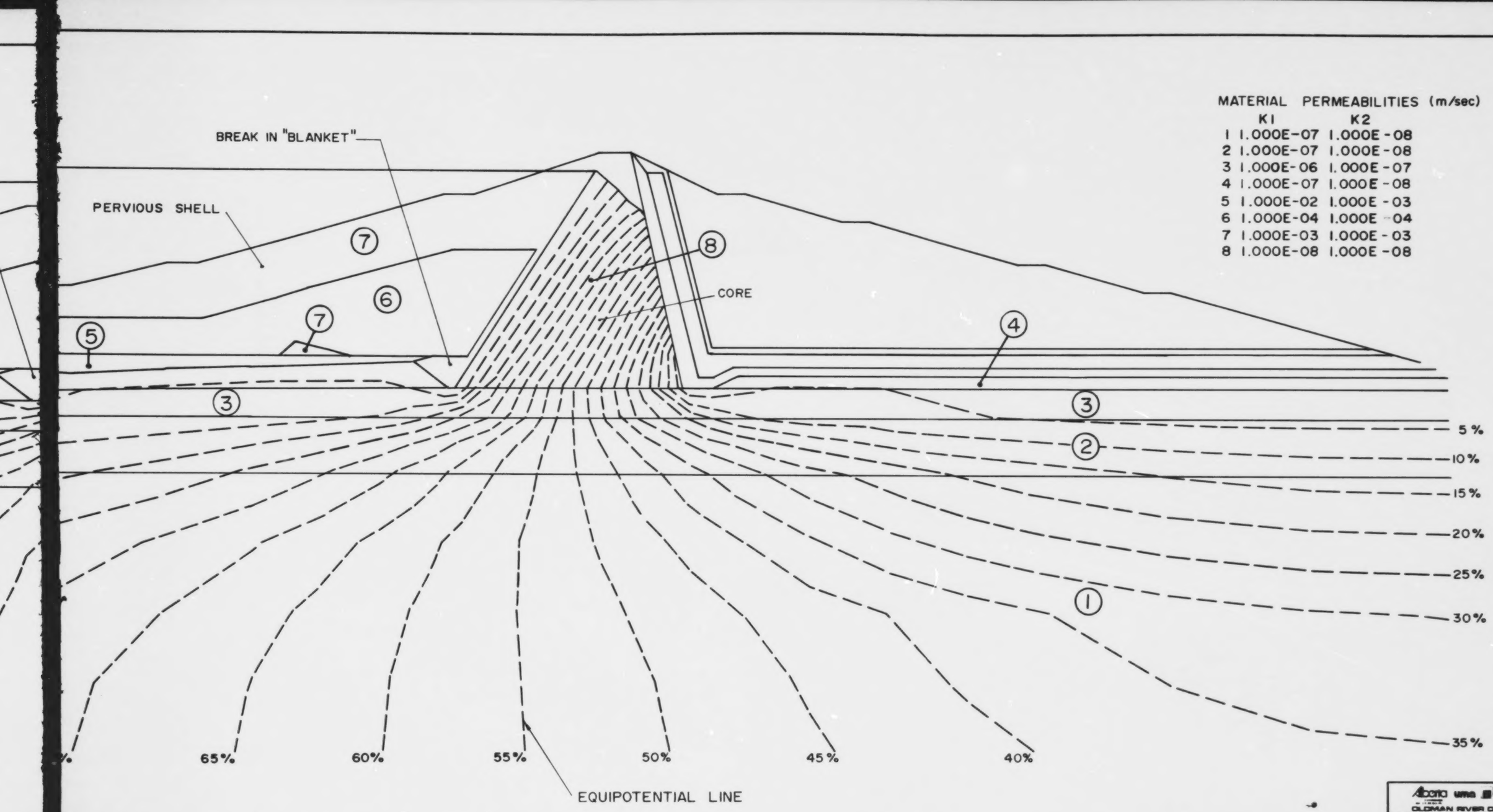
DAM - SECTION E
SH 1

PLATE E-12


EL 1118.6 F.S.L.
BREAK IN "BLANKET"
PERVIOUS SHELL
NATURAL "BLANKET"
ELEVATION (m)
1120
1100
1080
1060
1040
1020
1000
980
960
940
920
900
95%
90%
85%
80%
75%
70%
65%
60%
1
2
3
4
5
6
7

MATERIAL PERMEABILITIES (m/sec)

	K1	K2
1	1.000E-07	1.000E-08
2	1.000E-07	1.000E-08
3	1.000E-06	1.000E-07
4	1.000E-07	1.000E-08
5	1.000E-02	1.000E-03
6	1.000E-04	1.000E-04
7	1.000E-03	1.000E-03
8	1.000E-08	1.000E-08



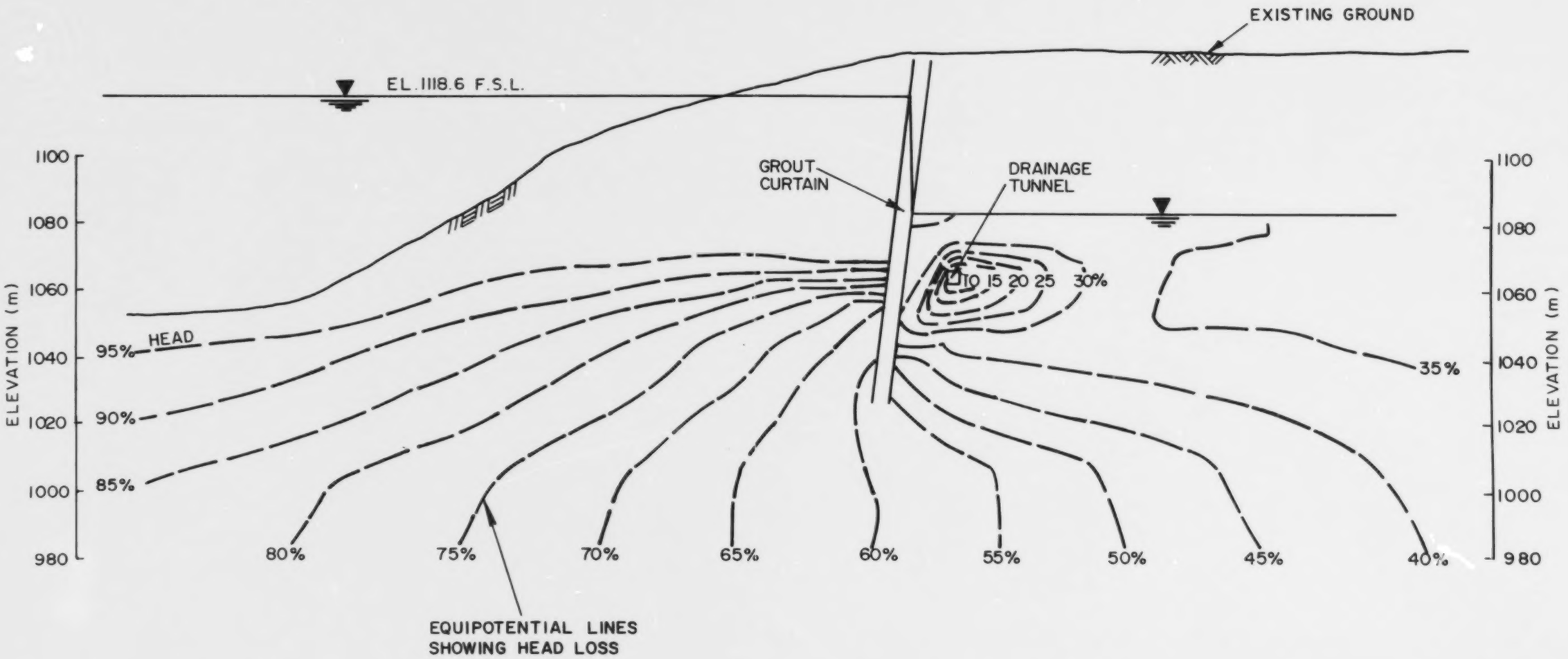
Alberta Power Limited
OLDMAN RIVER DAM

DAM – SECTION E
SH 2

PLATE E-13

MATERIAL PERMEABILITIES (m/sec.)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06



NOTE:

- GROUT CURTAIN TO EL 1020.0m
- DRAINAGE TUNNEL 16.0m DOWNSTREAM OF GROUT CURTAIN

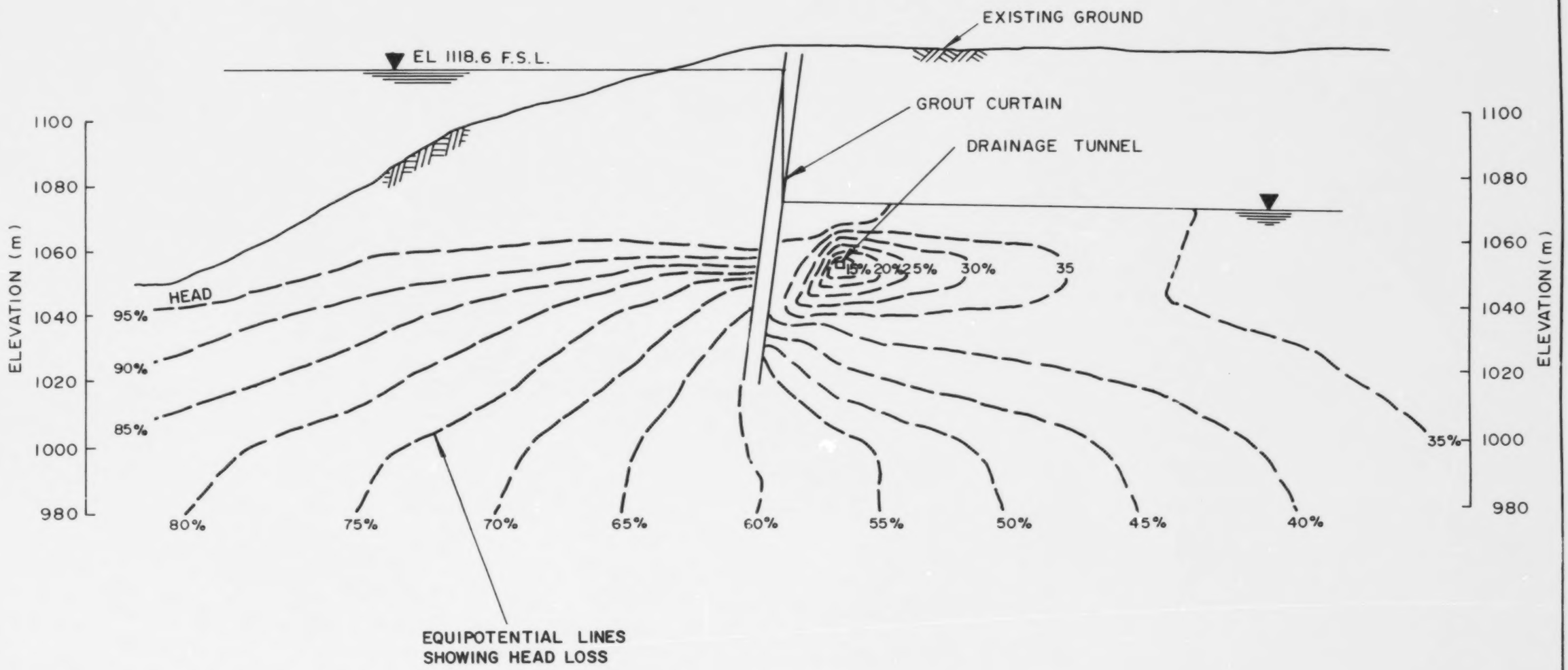
Acres International
OLDMAN RIVER DAM

LT. RIVER BANK - SECT. 'D'
DRAINAGE TUNNEL
LOCATION - 1

PLATE E-14

MATERIAL PERMEABILITIES (m/sec.)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06



NOTE:

- GROUT CURTAIN TO EL 1020.0 m
- DRAINAGE TUNNEL 21.0 m DOWNSTREAM OF GROUT CURTAIN

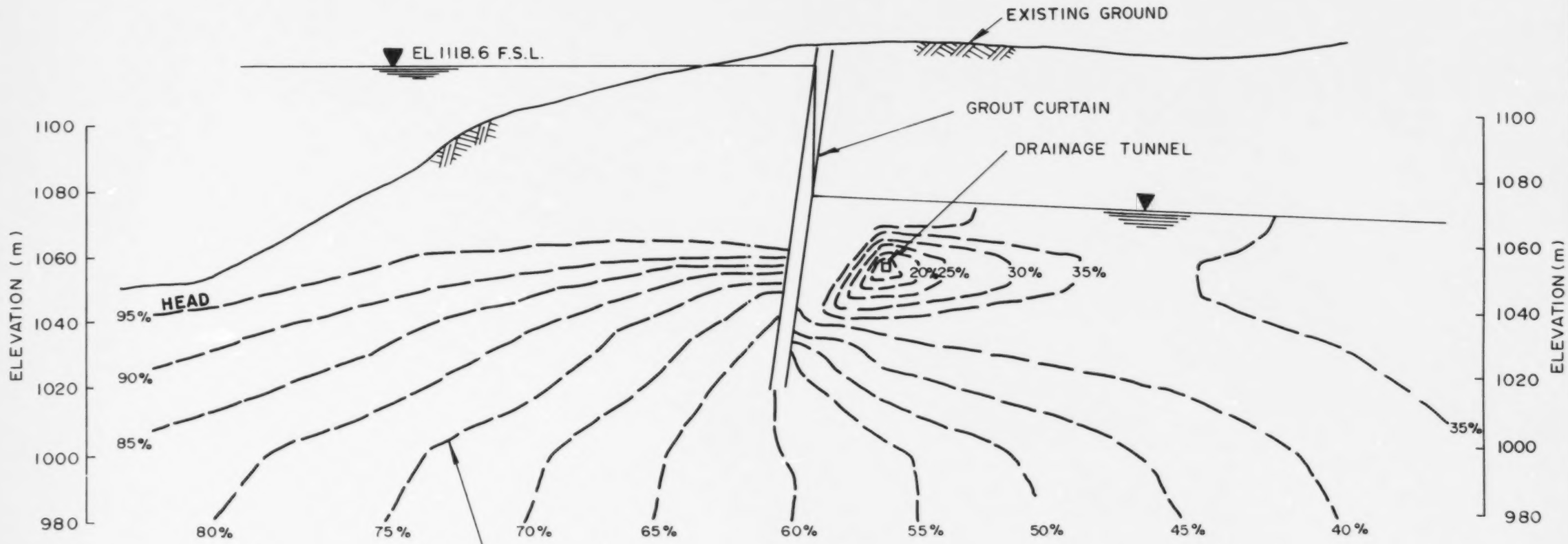
Alberta
OLDMAN RIVER DAM

LT. RIVER BANK - SECT. 'D'
DRAINAGE TUNNEL
LOCATION - 2

PLATE E-15

MATERIAL PERMEABILITIES (m / sec.)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06



NOTE:

- GROUT CURTAIN TO EL. 1020.0 m
- DRAINAGE TUNNEL 26.0 m DOWNSTREAM OF GROUT CURTAIN

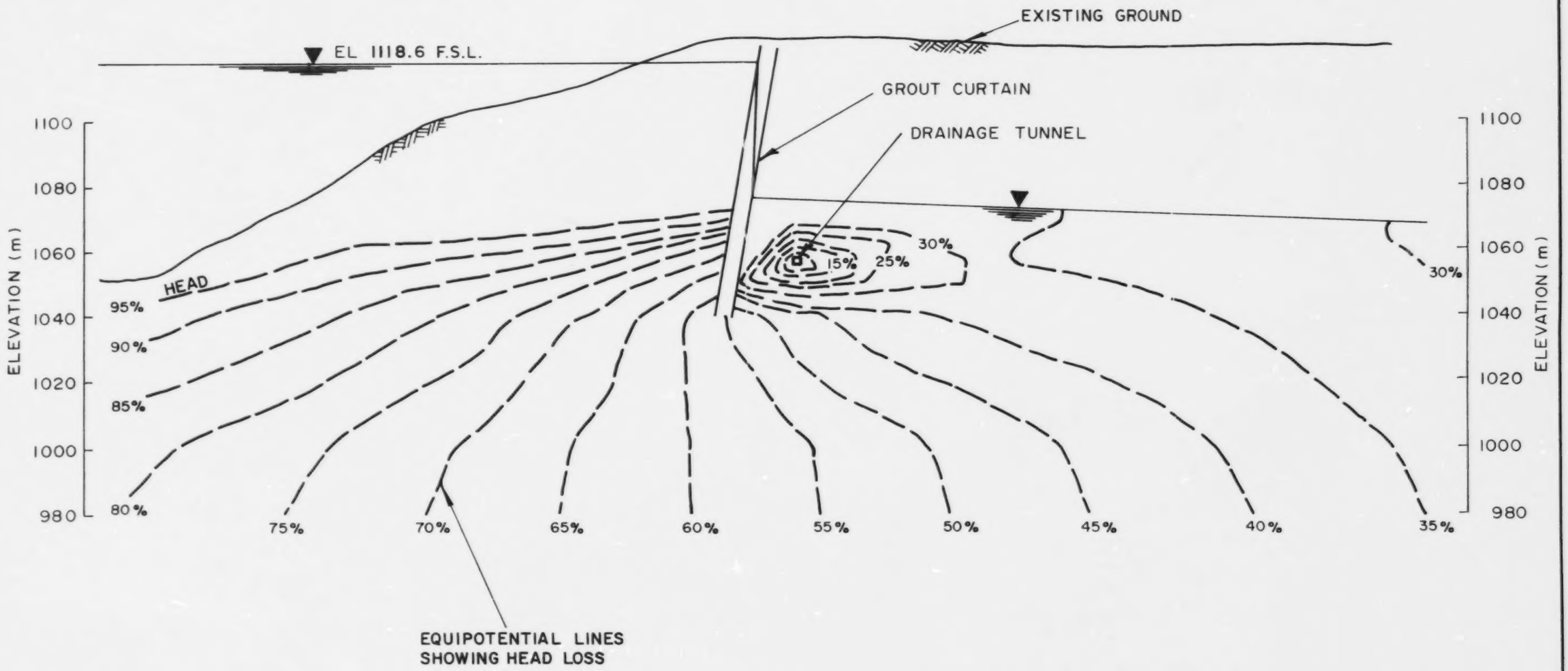
Alberta Utilities Commission
OLDMAN RIVER DAM

LT. RIVER BANK - SECT. 'D'
DRAINAGE TUNNEL
LOCATION - 3

PLATE E-16

MATERIAL PERMEABILITIES (m / sec.)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06



NOTE:

- GROUT CURTAIN TO EL 1040.0m
- DRAINAGE TUNNEL 16.0m DOWNSTREAM OF GROUT CURTAIN

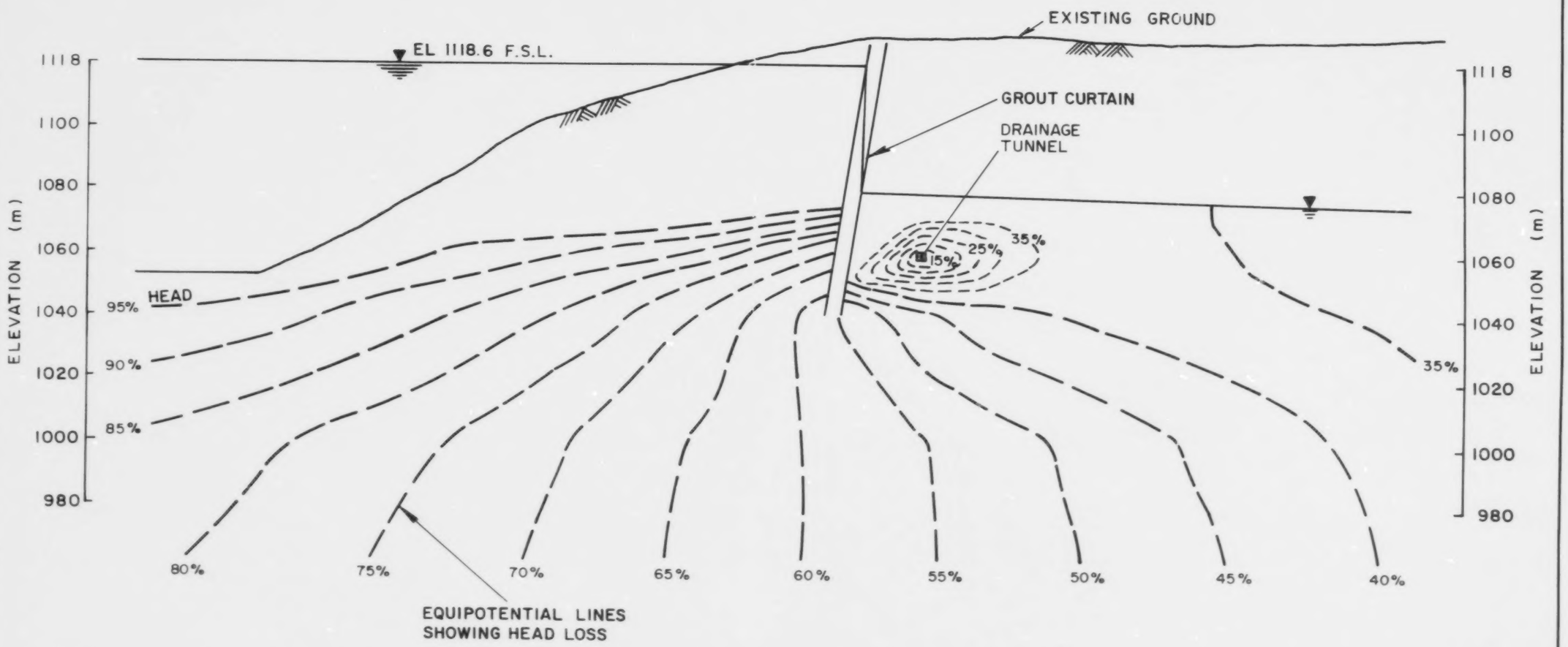
Alberta Power
OLDMAN RIVER DAM

LT. RIVER BANK - SECT. 'D'
DRAINAGE TUNNEL
LOCATION - 4

PLATE E-17

MATERIAL PERMEABILITIES (m/sec.)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06



NOTE:

- GROUT CURTAIN TO EL 1040.0 m
- DRAINAGE TUNNEL 21.0m DOWNSTREAM OF GROUT CURTAIN

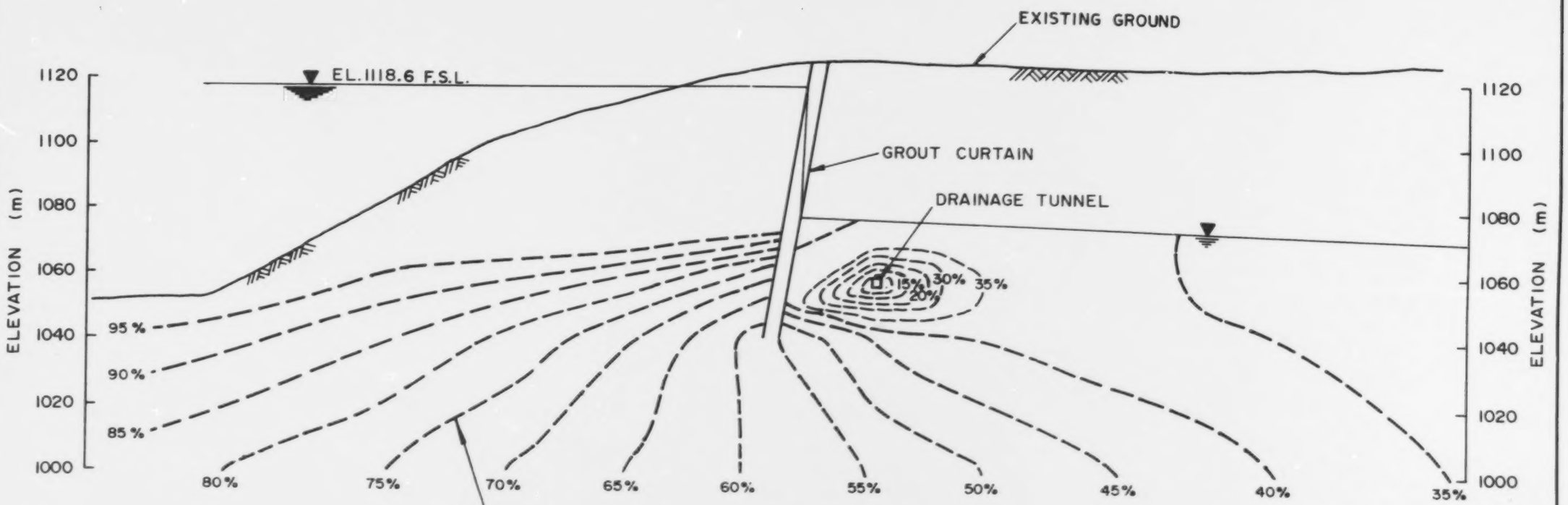
Alberta Utilities Commission
OLDMAN RIVER DAM

LT. RIVER BANK - SECT. 'D'
DRAINAGE TUNNEL
LOCATION - 5

PLATE E-18

MATERIAL PERMEABILITIES (m / sec.)

	K1	K2
BELOW EL. 1070.0	5.000E-07	5.000E-08
ABOVE EL. 1070.0	5.000E-05	5.000E-06



EQUIPOTENTIAL LINES
SHOWING HEAD LOSS

NOTE:

- GROUT CURTAIN TO EL 1040.0m
- DRAINAGE TUNNEL 26.0m DOWNSTREAM OF GROUT CURTAIN

OLDMAN RIVER DAM

LT. RIVER BANK - SECT. 'D'
DRAINAGE TUNNEL
LOCATION - 6

PLATE E-19

OLDMAN RIVER DAM

DESIGN REPORT

APPENDIX F

SLOPE PROTECTION

September, 1988
Updated June, 1989

APPENDIX F

SLOPE PROTECTION

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F-5 TYPICAL SECTION - BORROW AREA A

F.1 INTRODUCTION

Slope protection must be provided for all surfaces of the dam and dyke exposed to wind and wave erosion and to high velocities during major flood discharges through the spillway.

Factors that influence the design of the upstream slope protection are the windspeed, the direction of wind, the fetch or exposure, the slope of the embankment, the depth in front of the structure and the velocity of flood discharges. The quarried stone riprap that will be used to protect the upstream slopes has been designed according to the procedures described in the U.S. Corps of Engineers, "Shore Protection Manual" (USCE (1984)). The durability of the riprap is discussed in Appendix A.

The downstream toe of the dam will be armoured with quarried stone riprap to prevent erosion from back eddies that will pass across the face of the dam when the spillway is operating. The magnitude of the velocities are a function of the spillway discharge and the size of the plunge pool. The size of the riprap required to resist movement was determined as part of the hydraulic model testing of the spillway and plunge pool

The left abutment of the dam as well as the dyke lie downstream of Borrow Area "A" as shown on Plate F-1. It is possible that after reservoir impounding, erosion of the borrow pit slopes may undermine the long term stability and integrity of the dyke.

These three components of the slope protection design have been addressed in this appendix.

F.2 UPSTREAM SLOPE PROTECTION

F.2.1 Analysis of Wind

The wind gauge at Pincher Creek which has sixteen years of maximum hourly windspeed data has been used for this analysis. About eleven years of data were collected with a type 45B meter at a height of 8.8 m in the southwest corner of the town and about five years of data were collected by a type U2 meter at a height of 10 m at the airport about 6.5 km northwest of the town. The type 45B meter records a true maximum hourly windspeed whereas the type U2 meter tends to have more calm occurrences and more extreme occurrences.

The winds which would produce waves at the deep section of the dam come from the north and the winds which would produce waves at the dyke and the left bank freeboard section of the dam come from the 90 degree segment between west and north. This segment has the highest frequency of occurrence, with about 50 percent of the winds coming from this direction as shown in Plate F-2.

The upstream side of the dyke and the freeboard section of the dam faces north-west and are both aligned with their axes perpendicular to the general run of the existing Oldman River valley. There is a large fetch from this direction, which is also the direction from which the highest velocity winds with the greatest frequency of occurrence originate.

A log-normal frequency distribution of the maximum hourly windspeed from each direction for each month and for the open water season was completed by Alberta Environment. Wave generation equations are based on wind measurements made at 10 m above the ground surface. The windspeed data used in the frequency analysis was not adjusted to this level as is usual practice. The error introduced was estimated to be about 2 percent. As this error is well within the acceptable accuracy the wind frequency analysis was not recomputed. The results of the frequency analysis for the open water season are presented in Table F-1 and Plate F-3.

TABLE F-1

FREQUENCY ANALYSIS OF
ONE HOUR WINDSPEEDS
AT PINCHER CREEK

Return Period (years) / Wind Direction	Windspeed (Km/hr) 2	5	10	25	50	100
N	33	50	62	78	91	104
NE	30	37	42	47	51	55
E	27	31	34	36	38	40
SE	26	32	36	40	43	45
S	34	42	46	51	55	59
SW	80	97	107	119	128	136
W	81	100	111	125	135	145
NW	40	52	60	70	78	85
ALL	86	104	115	128	137	146

The windspeeds given in Table F-1 were adjusted by multiplying by appropriate factors for the air to water stability, for the effects of location and for the duration required for maximum wave generation. If the air is cooler than the water the boundary layer is unstable and the windspeed is more effective in causing wave growth. An air to water stability adjustment factor of 1.1 was used to account for these effects.

A location adjustment is required to account for the effect the surface roughness has on the wind profile. At high reservoir levels the water surface is relatively smooth compared to the topography near the wind gauge at Pincher Creek and therefore the windspeeds over the water would be higher than those recorded at the gauge. At low reservoir levels the effects of the river banks would result in lower windspeeds over the water than those recorded at the gauge. This adjustment factor varied from a minimum of 0.8 at the toe of the dam to a maximum of 1.2 at elevation 1,120 m.

A duration adjustment is required to account for the fact that windspeeds of an equal return period will be higher for a short duration than they would be for a longer duration. The durations required for maximum wave generation varied from 15 to 40 minutes depending on the location. The duration adjustment factors to the one hour windspeeds to give the required duration windspeeds ranged from about 1.1 to just over 1.0.

The wave growth formulas that were used for wave height predictions are expressed in terms of a wind stress factor. Wind generated waves grow as a result of the flux of momentum and energy from the air above the waves into the wave field. The wave growth formulas consider that this input of energy and momentum depends on the surface stress, which is highly dependent on the windspeed and other factors that describe the atmospheric boundary layer above the waves. After the other adjustments were made to the design windspeeds, the windspeeds were converted to a wind stress factor.

F.2.2 Fetch Calculations

The fetch for the various exposure locations in the reservoir was calculated by arithmetically averaging the distances obtained along 9 radial lines projected at 3 degree intervals from the point of interest. This method of fetch calculation is recommended in the 1984 revision of the Shore Protection Manual (USCE (1984)) as, use

of the more common "effective fetch", which uses the cosine rule, would underestimate actual wave development when used with the new wave growth equations. The deep section of the dam has a maximum fetch from the north of 3.5 km at elevation 1,125.5 m which reduces to a minimum fetch of 0.6 km at elevation 1,064.0 m. The dyke and the freeboard section of the dam have a fetch of 3.0 km from the west and 2.2 km from the north-west at full supply level.

F.2.3 Reservoir Levels

The reservoir full supply level is at elevation 1118.6 m which will only be exceeded during a flood greater than the 1:1000 year event. The dam crest level has been set at elevation 1125.5 m to provide flood storage during such rare floods including the probable maximum flood. The reservoir would rise to elevation 1124.5 m during these floods leaving 1 m of freeboard. During normal reservoir operation, the wave runup caused by the 1:100 year wind would be 2.2 m which is much less than the 6.9 m of freeboard provided at full supply level for flood storage.

During the early years of reservoir operation the water level will probably remain close to the full supply level and the erosive action of waves is likely to be centered between elevations 1115 m and 1120 m. As the reservoir becomes stressed by downstream demand the probability that the reservoir will be held at the full supply level will decrease. Thus during the open water season as the reservoir is drawn down to meet demand, different levels of the upstream face of the dam and to a lesser extent, of the dyke will be exposed to wave action. Using various scenarios of reservoir demand and operation the probability that the water level will lie within different ranges of elevation was estimated. Table F-2 shows the maximum value of probability obtained from the scenarios considered. The table shows that the reservoir level will lie between 1110 m and full supply level for most of the life of the project. Erosion of the upstream face due to the wave action in this range is likely to be significantly greater than lower down the slope.

TABLE F-2

MAXIMUM PROBABILITIES OF RESERVOIR LEVELS

Range of Reservoir Levels (m)	Maximum Probability of Reservoir Level being within the Range (%)	Windspeed Probability (%)	Windspeed Return Period (yr)	Combined Probability (%)
1125.5 - FSL	<0.1	1.0	100	<0.001
FSL - 1110.0	91	1.0	100	0.9
1110.0 - 1100.0	25	4.0	25	1.0
1100.0 - 1085.0	27	3.7	27	1.0
1085.0 - 1075.0	4	25.0	4	1.0
1075.0 - 1060.0	13	7.7	13	1.0

For each range of reservoir level, there is a different fetch because of the shelter afforded by the steep river valley banks.

F.2.4 Riprap Design

The probability of the reservoir being within a given range of elevation and the probability of a given windspeed were combined to obtain a combined probability of 1 percent i.e. 100 year return period. The riprap slope protection for the upstream face of the dam and the dyke below elevation 1120 m was designed for this combined probability of 1 percent. Above elevation 1120 m the riprap slope protection was selected considering the size of the structure, the ease of access for inspection and maintenance, and the probability of a high wind occurring during the period that the reservoir is surcharged to pass a flood exceeding the 1:1000 year event.

The wind stress factor for the duration required for maximum wave generation for a given fetch, wind direction and reservoir level was used to generate the significant wave height, period and wavelength at the various locations on the dam and dyke. The significant wave height is the average of the highest third of all waves. However the design wave height was selected as the average of the highest ten percent of all waves.

The maximum breaker height was calculated to determine the maximum wave height that could arrive at the dam and at the dyke. The maximum breaker height is the highest wave that can reach the structure just as the wave becomes unstable and breaks. It is dependent on the depth of water at the structure toe, the slope on which the structure is built, the incoming wave steepness and the distance the wave travels during breaking. For the dyke the design wave height of 3.6 m could not arrive at the dyke without first breaking. The maximum breaker height of 2.9 m was used for design. For the dam at full supply level the design wave height of 2.2 m could arrive without breaking and it was used for the design of the riprap for the top portion of the dam. Correspondingly lower maximum wave heights of 1.3 m and 0.5 m were determined for the design of the middle and bottom portions of the dam respectively. Neither of these waves would break before arrival.

The average weight (W_{50}) of stone required for stability of the riprap was calculated for various locations on the dam and the dyke using the Hudson formula for graded riprap cover layers. The recommended maximum weight of a graded riprap is four times the weight of the average or 50 percent size in the gradation and the minimum weight of a graded riprap is one eighth the weight of the 50 percent size in the gradation. The thickness of the riprap layer is made equal to two layers of the 50 percent size in the gradation. The bedding layer thickness is one half as thick as the riprap layer. The calculated weights for the different zones to be

protected were grouped into a limited number of ranges of stone size that could be conveniently specified. Gradation envelopes were then developed for each class of riprap using the design gradation as the minimum side of the envelope. The gradation envelopes for the various zones of riprap for the dam and dyke established are as follows:

Riprap Class A

Mass of specified individual rock particles (kg)	Percentage of the total mass to contain rock particles each with a mass greater than the specified individual rock particle mass	
	Upstream Face of Dyke	Downstream Toe of Dam
14,000	0	0
10,000	0 - 15	0 - 15
6,000	0 - 30	15 - 30
4,500	20 - 50	35 - 50
3,000	35 - 65	50 - 65
1,700	50 - 80	65 - 80
900	65 - 95	80 - 95
450	80 - 100	95 - 100
150	95 - 100	99 - 100
10	99 - 100	-

Riprap Class B

Mass of specified individual rock particles (kg)	Percentage of the total mass to contain rock particles each with a mass greater than the specified individual rock particle mass
7,000	0
3,200	0 - 25
2,400	15 - 40
1,500	30 - 55
900	50 - 70
500	65 - 85
250	80 - 100
90	95 - 100
10	99 - 100

Riprap Class C

Mass of specified individual rock particles (kg)	Percentage of the total mass to contain rock particles each with a mass greater than the specified individual rock particle mass
1,300	0
600	0 - 35
350	20 - 50
190	45 - 62
90	60 - 80
30	80 - 100
10	90 - 100
2	99 - 100

Riprap Class D

Specified Sieve Size	Percent by Mass Finer than the Specified Sieve Size
450 mm	100
350 mm	70 - 100
300 mm	50 - 100
200 mm	35 - 50
150 mm	15 - 35
80 mm	0 - 10
40 mm	0 - 3

Riprap Bedding

Specified Sieve Size	Percent by Mass Finer than the Specified Sieve Size
450 mm	100
350 mm	30 - 100
300 mm	50 - 100
200 mm	30 - 100
150 mm	15 - 70
80 mm	0 - 40
40 mm	0 - 30
20 mm	0 - 10

F.3 DOWNSTREAM SLOPE PROTECTION

The riprap required for erosion protection of the downstream toe of the dam and the spillway approach channel were designed for the conditions obtaining during the probable maximum flood. The maximum velocities at the toe of the dam and in the spillway approach channel were measured in the 1:75 scale hydraulic model of the spillway (NHCL (1987) NHCL (1988)). The sizes of riprap that will remain stable on the central portion of the downstream toe of the dam and on each side while the spillway is discharging the probable maximum flood are 1200 mm and 80 mm respectively. In the spillway approach channel, the size of riprap that will remain stable is 500 mm. With the exception of the 1200 mm size, these riprap sizes are similar to those required for the face of the dam and dyke, and so no new gradation limits were created.

F.4 UNDERLAYERS

Based on discussions with engineers at the U.S. Corps of Engineers, the underlayer for the riprap was designed using a well graded material rather than a uniformly graded material as described in the "Shore Protection Manual" (USCE (1984)). The relationship between the gradations of the designed riprap, its underlayer or multiple underlayers and the fill material on the slope of the dam and dyke were determined by application of the filter criteria to prevent particle migration. For this design the filter criteria used was that the d_{15} size of any layer should not be greater than four times the d_{85} size of the underlying layer, and that the d_{50} size of the layer should not be greater than twenty-five times the d_{50} size of the underlying layer.

F.5 DESIGN

The upstream face of the dam was divided into four regions requiring each different riprap slope protection. The first region, from elevation 1120 m down to elevation 1110 m, will be subject to the most severe wave action and will be protected with a 1.5 m thick

layer of Class B Riprap underlaid by a 0.75 m thick layer of Zone 7 Riprap Bedding material. The second region from elevation 1110 m down to elevation 1085 m, will be subject to moderate wave action and will be protected with a 1.0 m thick layer of Class C Riprap underlaid by a 0.5 m thick layer of Zone 7 Riprap Bedding material. Below elevation 1085 m, the wave action will be mild and the slopes will be protected with a 0.5 m thick layer of Class D Riprap which will be underlaid by the Zone 5 Granular Fill material of the shell of dam.

As it is not possible to determine the magnitude or frequency of winds that might occur during the relatively short period that the reservoir will be surcharged above full supply level, immediately following the extreme 1:1000 year or larger storm the design of the riprap slope protection above elevation 1120 m cannot be rationalized in the same manner that was used below elevation 1120 m. The top of the dam being the major structure, will be protected by continuing the Class B riprap above elevation 1120 m up to he crest. This is a conservative approach as the design will provide protection up to the 1:100 year wind.

The left abutment of the dam will be protected with a 1.5 m thick layer of Class B Riprap underlaid by a 0.75 m thick layer of Zone 7 Riprap Bedding material from elevation 1120 m down to elevation 1110 m. Above elevation 1120 m the slopes are very flat and will not be protected with riprap. Any slopes that are below elevation 1120 m and are flatter than 7 horizontal to 1 vertical will be protected with a 0.5 m thick layer of Zone 7 Riprap Bedding material underlaid by a 0.5 m thick layer of Zone 5 Granular Fill material . From elevation 1110 m down to elevation 1085 m, the left abutment slopes will be protected with a 1.0 m thick layer of Class C Riprap underlaid by a 0.5 m thick layer of Zone 7 Riprap Bedding material.

The riprap slope protection required for the downstream toe of the dam consists of a 2 m thick layer of Class A Riprap underlaid by a 1.0 m thick layer of Zone 7 Riprap Bedding material in the central portion and a 2m thick layer of Class B Riprap material underlaid by a 1.0 m thick layer of Zone 7 Riprap Bedding material on each side. The spillway approach channel will be protected with a 1 m thick layer of Class C Riprap underlaid by a 0.5 m thick layer of Zone 7 Riprap Bedding material.

The low head dyke, with its long freeboard sections above full supply level and the freeboard section of the dam on the left abutment are easily accessible for inspection and repairs and need not be protected so conservatively as for the main section of the dam. Class B Riprap will therefore be provided above elevation 1120 m. With this protection, some damage to the slope protection might take place if the winds exceed the 1:7 year event. The combined probability of a storm greater than the 1:1000 year event followed by an hourly wind greater than the 1:7 year event occuring during the maximum two day period (for the probable maximum flood) that the reservoir level would be above FSL, must be very low. Implicit in the design is the risk and cost of possible future remedial work.

The riprap slope protection for the upstream face of the dyke below elevation 1120 m will consist of a 2 m thick layer of Class A Riprap underlaid by a 1 m thick layer of Zone 7 Riprap Bedding material.

Table F-3 and Plate F-4 summarize the weight, size, thickness and gradation of the riprap and underlayers required to provide slope protection for various locations of the Oldman River Dam Project.

F.6 BORROW AREA "A" PROTECTION

Wave erosion of the slopes of the pit excavated in Borrow Area "A" to obtain dam fill material is of concern because of its proximity to the upstream toe of the dyke. Any erosion occurring in the pit slopes could influence the stability of the dyke. Test hole data

available in and around the proposed borrow area reveal up to 11 m of glaciolacustrine silt and clay overlying the clay and silty clay tills that will be used to construct the core of the dam.

The glaciolacustrine clay will be exposed in the borrow pit slopes both above and below FSL. The maximum exposure of this portion of the reservoir to erosion is the zone between elevation 1110 m and the full supply level.

Shear stability analysis of the slopes indicate that the maximum overall stable slope in the glaciolacustrine silt and clay and the clay and silty clay tills immediately upstream of the dyke is 7 horizontal to 1 vertical. This slope was designed to be stable under rapid drawdown conditions, but does not account for the effects of wave generated erosion. The dyke slopes of 4 horizontal to 1 vertical will be stable and protected with riprap.

A design rationale to assess the stability of the excavated slopes of the borrow pit against wave action was not easy to develop as wave erosion of shorelines consisting of cohesive soils is a very poorly documented and understood phenomenon. The process of erosion of cohesive soils is much different than that of granular shorelines for which much data are available.

A literature review was carried out on the subject of erosion of cohesive soils indicated that most of the laboratory research into the erosion of cohesive soils has been directed to stream flow action rather than wave action (Andres (1985), Chapuis (1986), Kandiah (1974)).

Acres (1979) studied the effects of wave action on a reservoir at Arnprior, Ontario and concluded that, from an analytical point of view, a realistic prediction of erosion damage to cohesive slopes of a reservoir is not possible with the present state of the art.

TABLE F-3

SUMMARY OF SPECIFIED SLOPE PROTECTION ZONES

Location	Design	Specified Riprap				Bedding Layer			
	W_{50} (kg)	Zone	W_{50} (kg)	D50 (mm)	Thickness (m)	Zone No	W_{50} (kg)	D50 (mm)	Thickness (m)
Dam (U/S face)									
1125.5 - 1110.0	874	8B	900	800	1.50	7	2	100	0.75
1110.0 - 1085.0	134	8C	150	450	1.00	7	2	100	0.50
1085.0 - toe of dam	8.6	8D	10	200	0.50	-	-	---	----
Dam (left abutment)									
1120.0 - 1110.0	874	8B	900	800	1.50	7	2	100	0.75
1110.0 - 1085.0	134	8C	150	450	1.00	7	2	100	0.50
slope < 7:1	(3)	7	2	100	0.50	5	-	30	0.50
Dam (D/S face)									
1065.0 - toe of dam (centre)	(4) 1200 1430	8A	3000	1200	2.00	7	2	100	1.00
1065.0 - toe of dam (each side)	-	8B	900	800	2.00	7	2	100	1.00
Dyke									
1125.5 - 1120.0	(6)	8B	900	800	1.50	7	2	100	0.75
1120.0 - toe of dyke	1644	8A	1700	990	2.00	7	2	100	1.00
Spillway Approach Channel									
1120.0 -bottom of channel	(4) 480-600	8C	150	450	1.00	7	2	100	0.50

NOTE:

(1) The D_{50} size has been calculated from the formula:

$$D_{50} = 1.15\left[\frac{(W_{50})}{(2650)}\right]0.33$$

and has been rounded upwards to the next convenient size. Plate F-4 shows these gradations based on size. The riprap supply contracts specified the gradation by mass not size.

(2) Layer thickness is based on specified gradation not design gradation.

(3) Nominal sizing based on beaching action.

(4) Range of stone sizes in mm that the hydraulic model test showed to just stable during PMF flow conditions (NHCL (1987)).

(5) Class B riprap was selected for each side of the downstream toe of the dam based on the hydraulic model test of PMF flow conditions (NHCL (1988)).

(6) Class B riprap was selected for the zone above elevation 1120 m on the dyke, as appropriate protection against an extremely rare event.

Mollard (1986) studied natural and manmade reservoirs in the Prairie Provinces by examining air photos taken over a 30 year period and has reported on erosion related loss of land around the edge of reservoirs. Mollard cites rates of erosion ranging between 1 and 12 m/year with 1 to 3 m/year being common. Shoreline erosion is very site specific and often only those areas of major erosion are reported. Many areas around most lakes suffer little erosion.

The erosion process results in the formation of a beach or foreshore slope. Mollard cites beach slopes that range between 6.5 horizontal to 1 vertical and 15 horizontal to 1 vertical for tills and 30 horizontal to 1 vertical or flatter for glacial lake clays.

It is evident that there is no way to technically assess the extent of erosion that can be expected upstream of the Oldman River dam and dyke. Furthermore, only broad conservative estimates of long term foreshore slopes are presented in the literature.

It is noted that significant wave action can be expected on the reservoir and that it will cause erosion, beach formation and steep scarps along the high water line. This action is expected to be most noticeable during the early life of the reservoir but it should reduce in severity with time as the slopes stabilize.

In order to preserve the stability of the dyke and the freeboard section of the dam, the following alternative designs were considered:

I Accept the very flat 15 horizontal to 1 vertical and 30 horizontal to 1 vertical slopes presented in the literature that might result from erosion. This will effectively make exploitation of Borrow Area "A" impracticable as it would be impossible to recover much of the glacial till without removal of extensive volumes of the glaciolacustrine silt and clay deposits.

II Develop the borrow area close to the upstream toe of the dyke and provide slope protection to the resulting excavated face to prevent future erosion.

III Provide a 150 m setback distance from the toe of the dyke to the top of the stable slope of the borrow pit so that the overall slope will be 15 horizontal to 1 vertical. This will reduce to an acceptable level the risk of future danger to the dyke. The actual progress of erosion during the early years of the project will be observed and remedial action will be taken if necessary.

Alternative III was selected because a minimum additional initial capital cost of approximately $2,000,000 is estimated for Alternative II. Alternative I is unacceptable. Future maintenance costs may be incurred if detrimental erosion occurs. Remedial measures could include riprap slope protection or rebuilding the dyke or freeboard section of the dam further downstream. Alternative III is illustrated in Plate F-5.

In some areas close to the left abutment of the dam, the existing slopes of the river bank from the surface of bedrock to the upstream toe of the dyke and the freeboard section of the dam, are steeper than 15 horizontal to 1 vertical. These slopes will be protected with riprap.

F.7 SUMMARY AND CONCLUSIONS

The upstream slopes of the dam, dyke and left abutment of the dam can all be protected from wind and wave erosion by using riprap slope protection. The durability of the riprap is discussed in

Appendix G. The riprap design will provide protection below elevation 1120 m for a combined probability of wind and reservoir level of 1 percent which is for a 100 year return period.

Borrow Area "A" will be developed such that a 7 horizontal to 1 vertical slope from the top to the base of the borrow area will define the limits of the pit base. A minimum 150 m setback will be provided between the upstream toe of the dyke and the top of the borrow pit slope.

The riprap slope protection of the downstream toe of the dam and the spillway approach channel were designed for the probable maximum flood. The maximum velocities and sizes of the riprap were determined in the spillway model study.

With adequate observation and maintenance, the slope protection for the Oldman River Dam Project will perform satisfactory.

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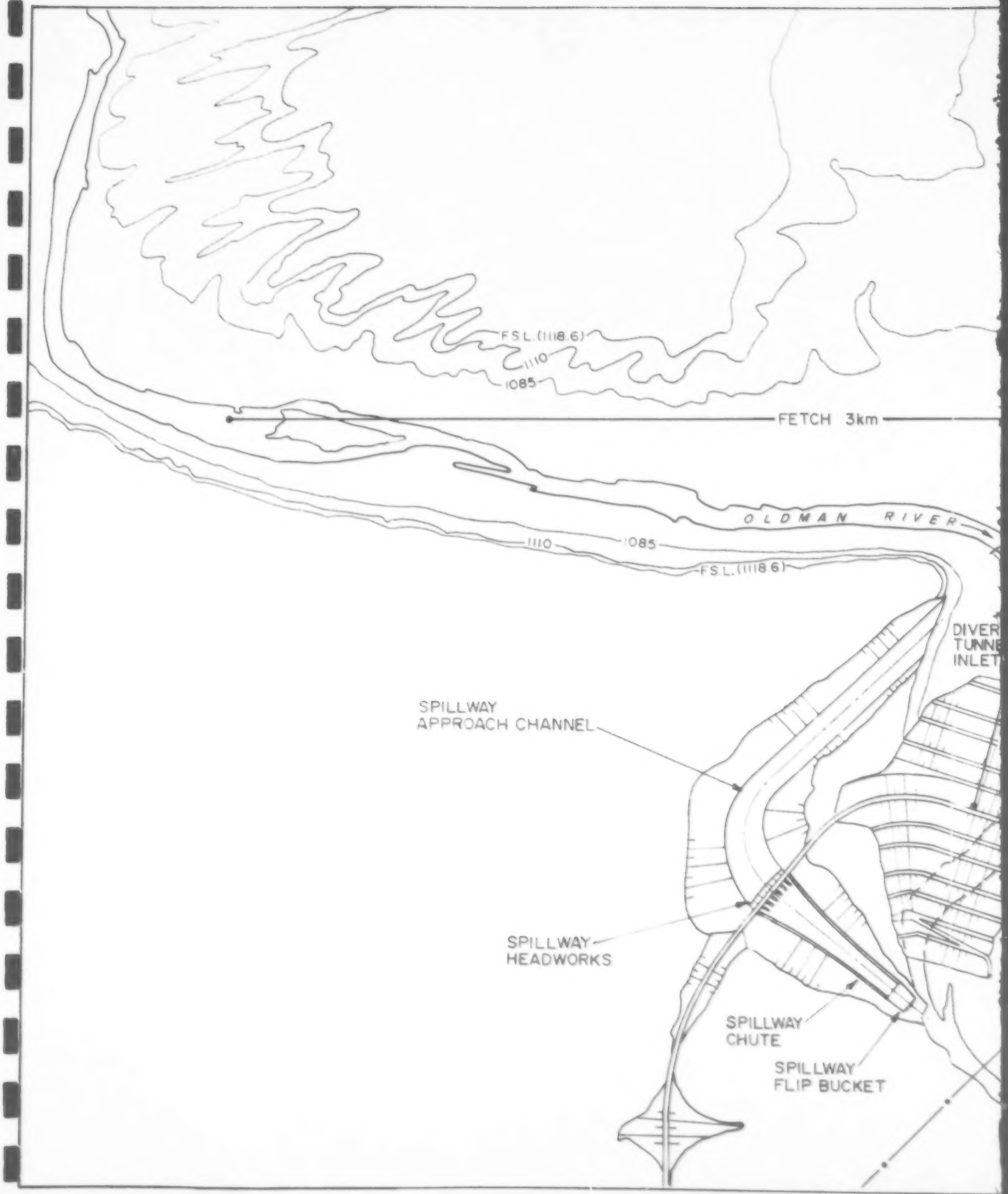
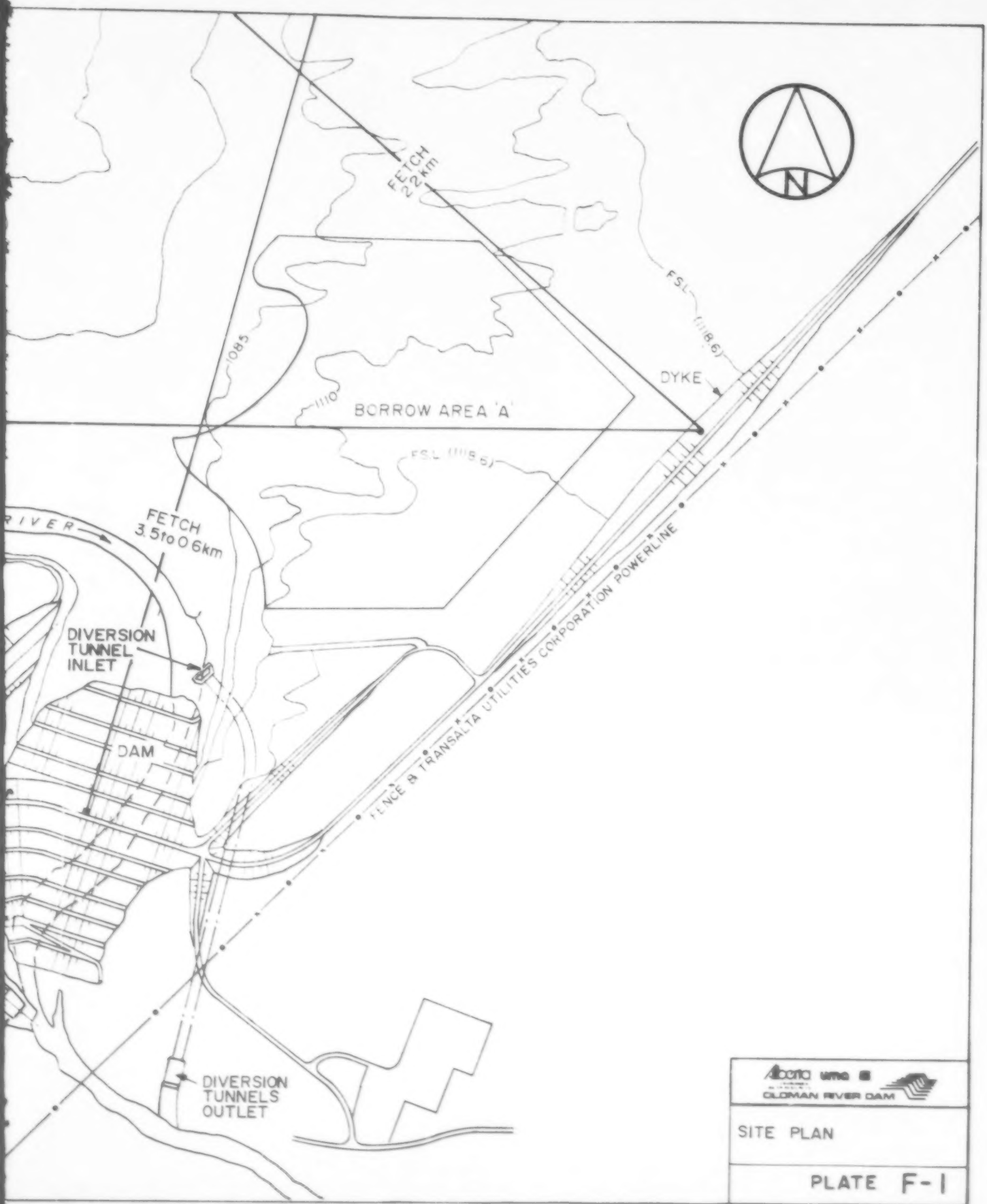
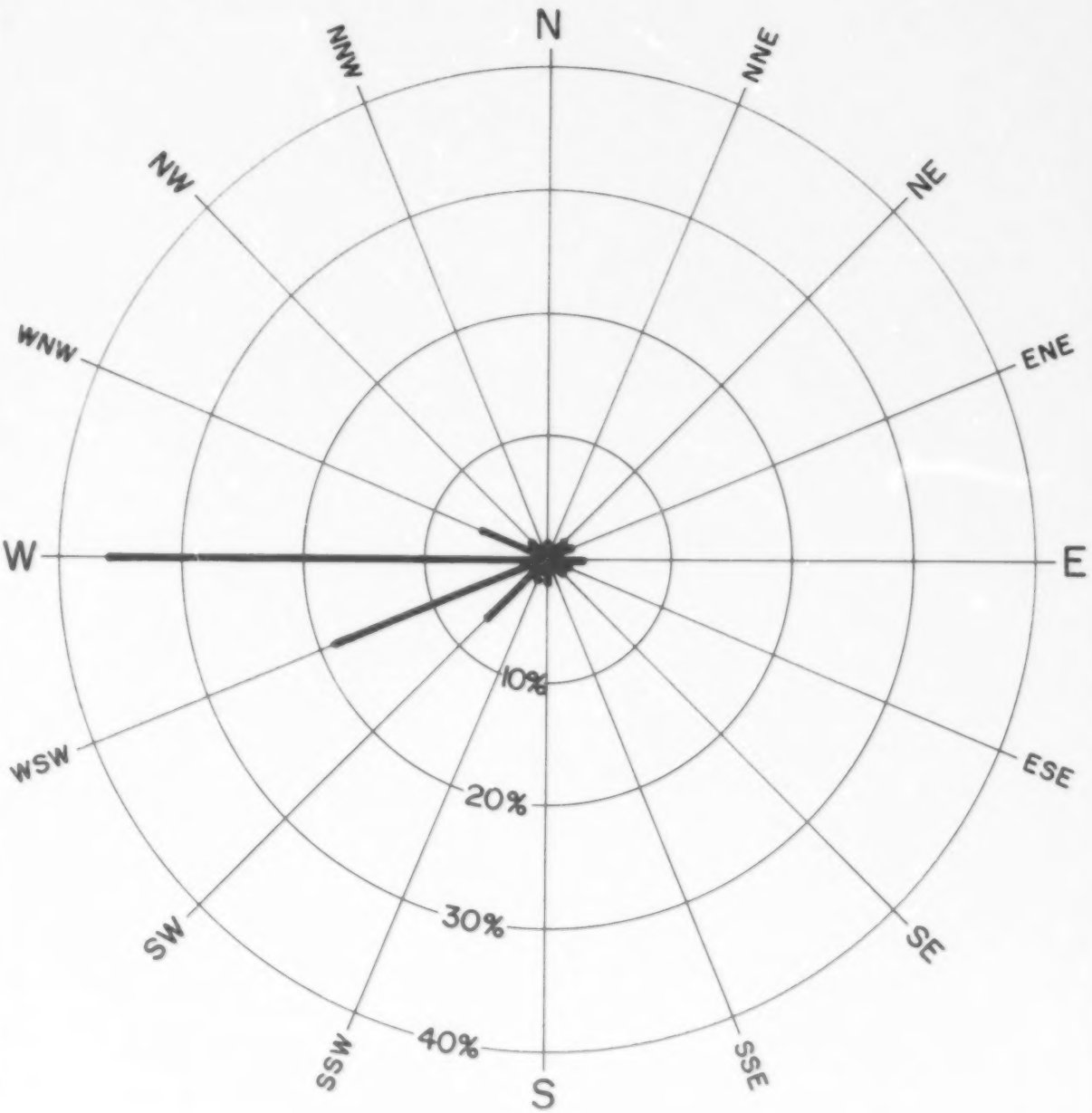

F.S.L. (1118.6)
1110
1085
FETCH 3km
OLDMAN RIVER
1110
1085
F.S.L. (1118.6)
DIVER
TUNNE
INLET
SPILLWAY
APPROACH CHANNEL
SPILLWAY
HEADWORKS
SPILLWAY
CHUTE
SPILLWAY
FLIP BUCKET

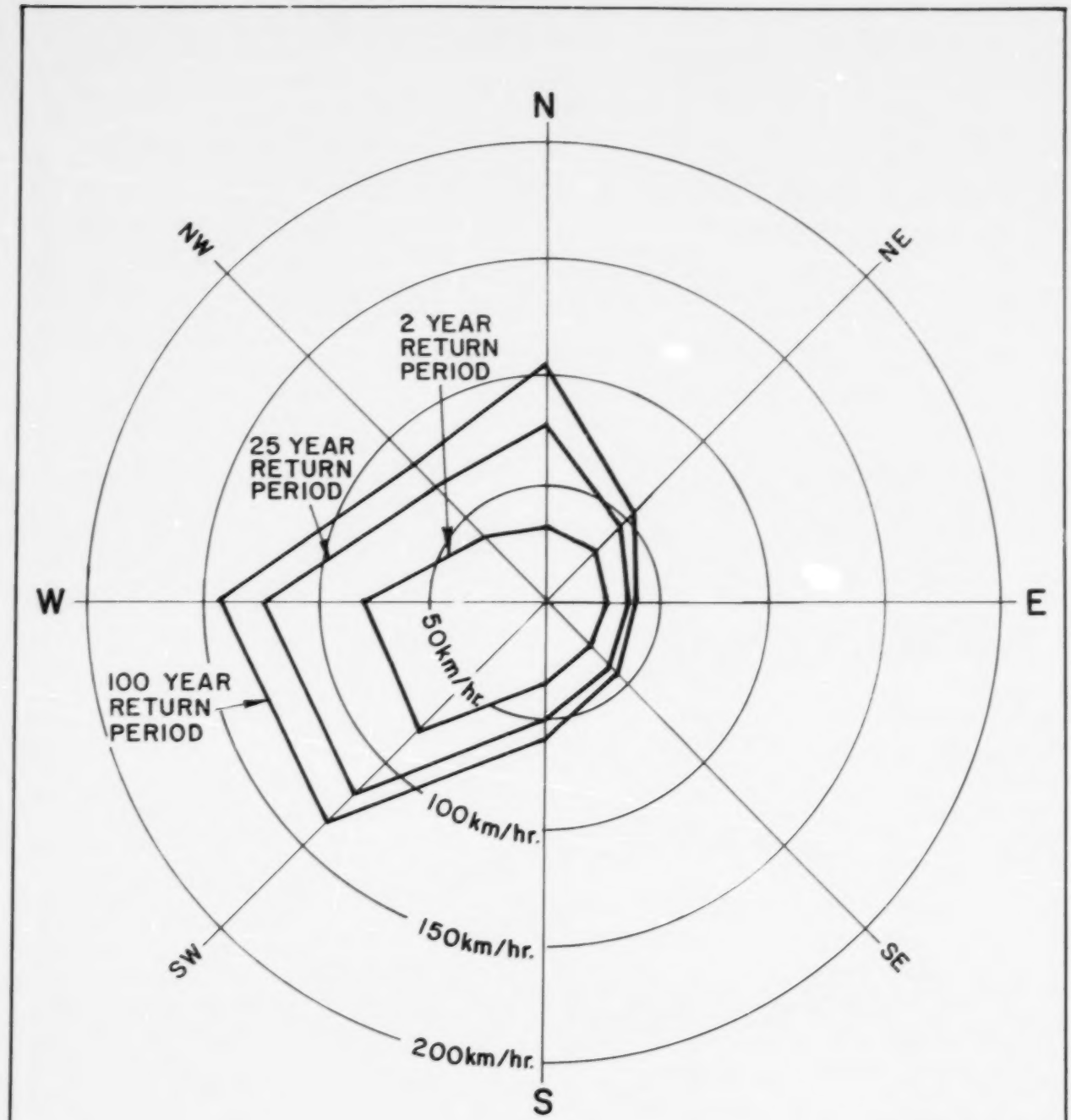

FETCH 2.2km
N
FSL (1118.6)
1085
1110
DYKE
BORROW AREA 'A'
FSL (1118.6)
RIVER
FETCH 3.5 to 0.6km
DIVERSION TUNNEL INLET
DAM
FENCE & TRANSALTA UTILITIES CORPORATION POWERLINE
DIVERSION TUNNELS OUTLET
Alberta Power
OLDMAN RIVER DAM
SITE PLAN
PLATE F-1



Alberta Power Limited
OLDMAN RIVER DAM

WIND DIRECTION
FREQUENCY

PLATE F-2



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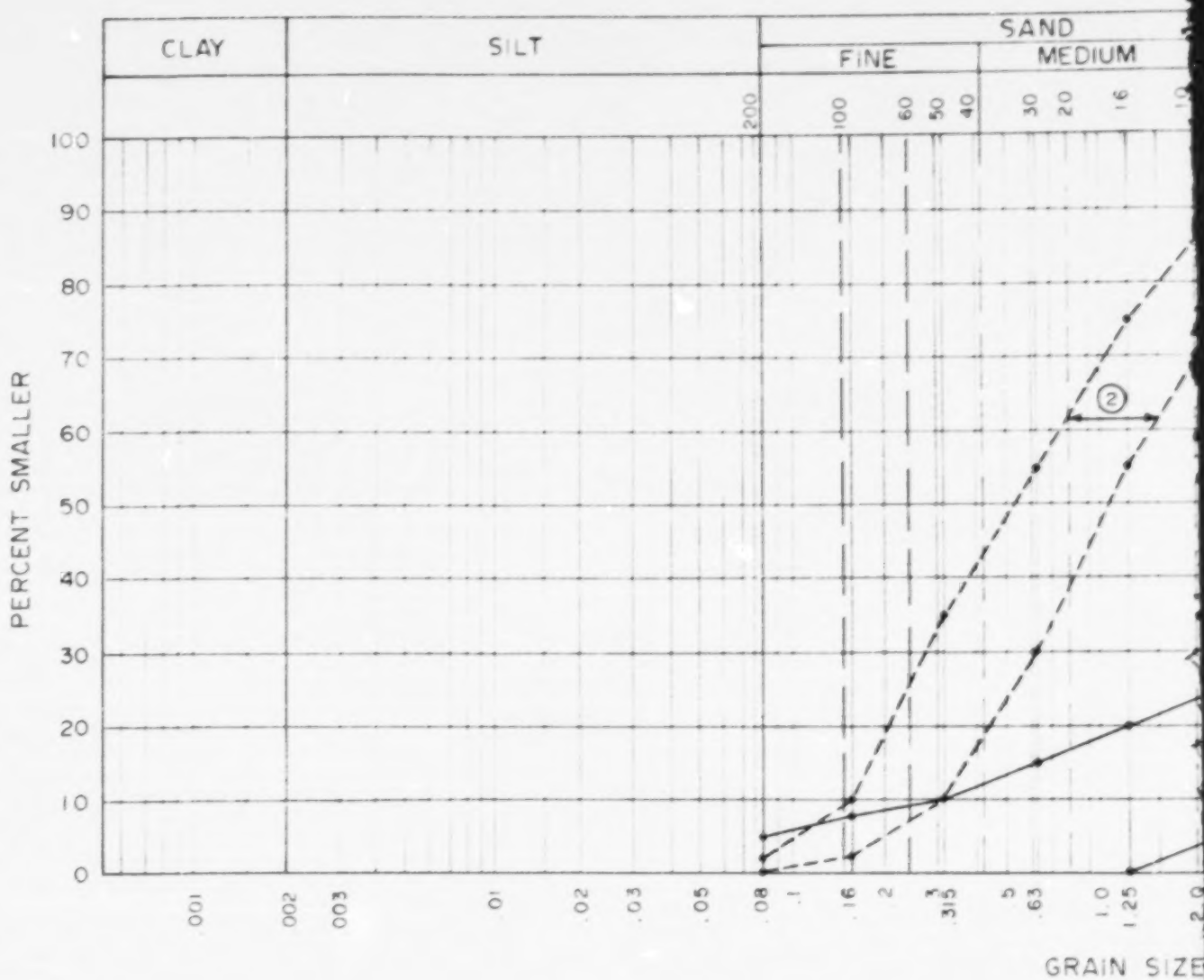
OLDMAN RIVER DAM

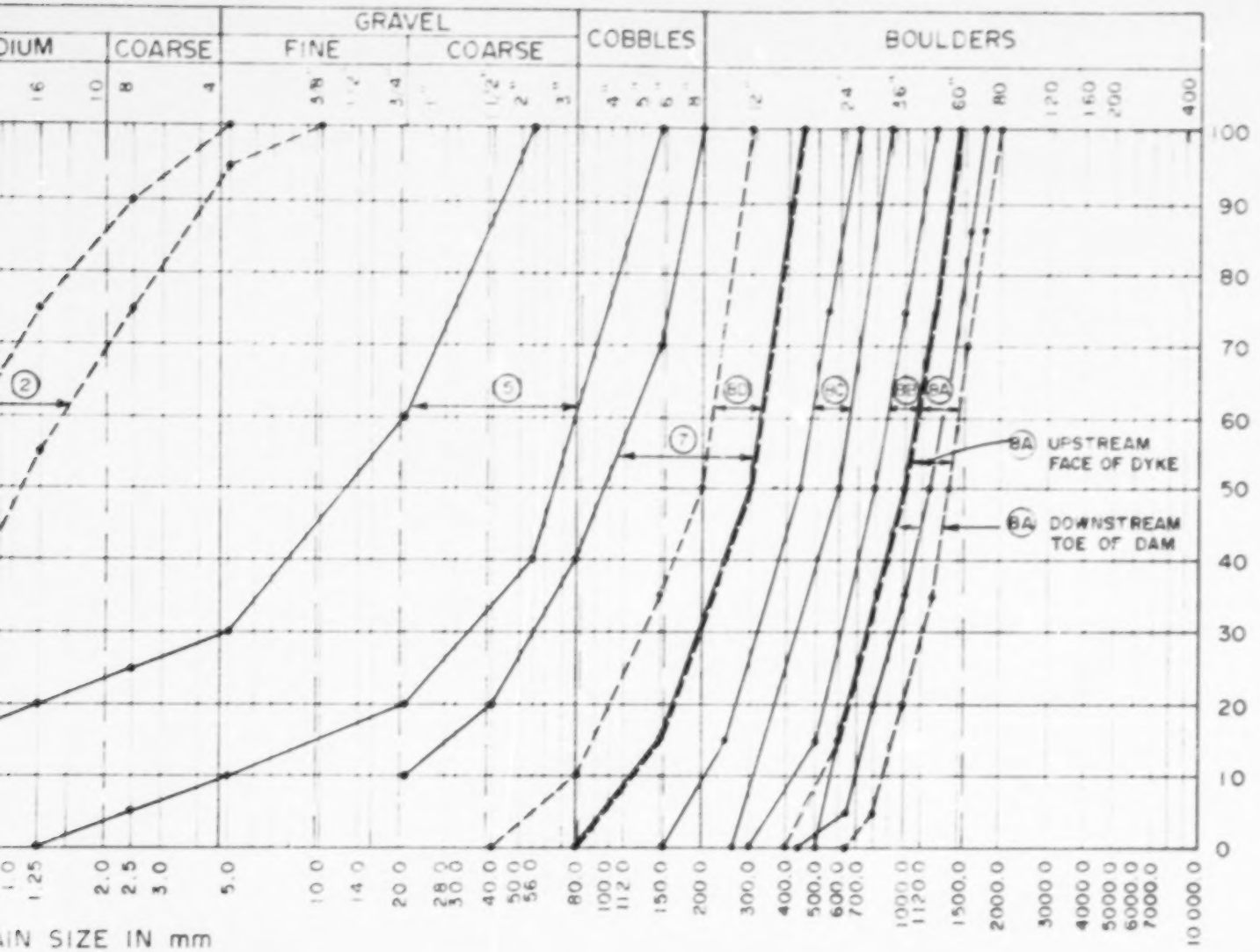
ONE HOUR
WINDSPEED FREQUENCY

PLATE F-3



LEGEND:

- ② FILTER
- ⑤ GRANULAR FILL
- ⑦ RIPRAP BEDDING
- ⑧A RIPRAP CLASS 'A'
- ⑧B RIPRAP CLASS 'B'
- ⑧C RIPRAP CLASS 'C'
- ⑧D RIPRAP CLASS 'D'

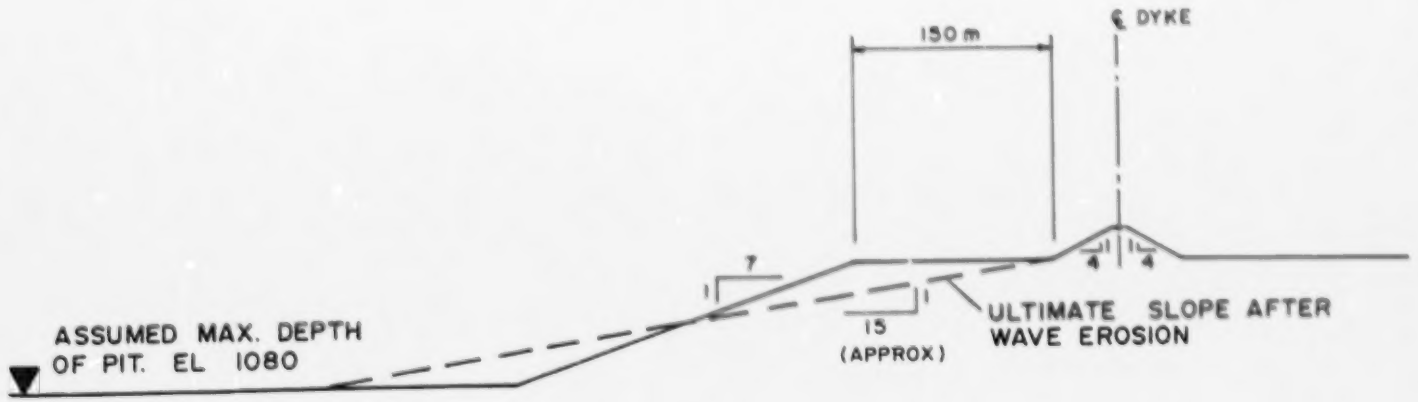


AIN SIZE IN mm

OLDMAN RIVER DAM

RIPRAP GRADATION

PLATE F-4



TYPICAL SECTION

SCALE 1:500 HORZ.
1:200 VERT.

Alcan uma
OLOMAN RIVER DAM

TYPICAL SECTION,
BORROW AREA 'A'

PLATE F-5